VOLUME 87 NO. WW1
FEBRUARY 1961

JOURNAL of the

Waterways and Harbors Division

PROCEEDINGS OF THE

AMERICAN SOCIETY
OF CIVIL ENGINEERS

BASIC REQUIREMENTS FOR MANUSCRIPTS

Original papers and discussions of current papers should be submitted to the Manager of Technical Publications, ASCE. Authors should indicate the technical division to which the paper is referred. The final date on which a discussion should reach the Society is given as a footnote with each paper. Those who are planning to submit material will expedite the review and publication procedures by complying with the following basic requirements:

1. Titles must have a length not exceeding 50 characters and spaces.

2. A summary of approximately 50 words must accompany the paper, a 300-word synopsis must precede it, and a set of conclusions must end it.

3. The manuscript (an original ribbon copy and two duplicate copies) should be double spaced on one side of 8½-inch by 11-inch paper. Three copies of all illustrations, tables, etc., must be included.

4. The author's full name, Society membership grade, and footnote reference stating present employment must appear on the first page of the paper.

5. Mathematics are recomposed from the copy that is submitted. Because of this, it is necessary that letters be drawn carefully, and that special symbols be properly identified. The letter symbols used should be defined where they first appear, in the illustrations or in the text, and arranged alphabetically in an Appendix.

6. Tables should be typed (an original ribbon copy and two duplicate copies) on one side of 8½-inch by 11-inch paper. Specific illustrations and explanation must be made in the text for each table.

7. Illustrations must be drawn in black ink on one side of 8½-inch by 11-inch paper. Because illustrations will be reproduced with a width of between 3-inches and 4½-inches, the lettering must be large enough to be legible at this width. Photographs should be submitted as glossy prints. Explanations and descriptions must be made within the text for each illustration.

8. The desirable average length of a paper is about 12,000 words and the absolute maximum is 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

9. Technical papers intended for publication must be written in the third person.

10. The author should distinguish between a list of "Reading References" and a "Bibliography," which would encompass the subject of his paper.

Reprints from this Journal may be made on condition that the full title of the paper, name of author, page reference, and date of publication by the Society are given. The Society is not responsible for any statement made or opinion expressed in its publications.

This Journal is published quarterly by the American Society of Civil Engineers. Publication office is at 2500 South State Street, Ann Arbor, Michigan. Editorial and General Offices are at 33 West 39 Street, New York 18, New York. $4.00 of a member's dues are applied as a subscription to this Journal. Second-class postage paid at Ann Arbor, Michigan.

The index for 1959 was published as ASCE Publication 1960-10 (list price $2.00); indexes for previous years are also available.

HW,IR,PO,SW.

Journal of the

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

WATERWAYS AND HARBORS DIVISION
EXECUTIVE COMMITTEE
Richard O. Eaton, Chairman; Evan W. Vaughan, Vice Chairman;
Joe W. Johnson; Charles F. MacNish; Robert J. Winters, Secretary
D. B. Ventres, Board Contact Member

COMMITTEE ON PUBLICATIONS
Jay V. Hall, Jr., Chairman; Eric Bottoms; Joseph M. Caldwell;
Ellsworth I. Davis; James W. Dunham; Donald R. Forrest
Richard O. Eaton, Division Executive Committee Contact Member

CONTENTS

February, 1961

Papers

Page

Small-Boat Harbor Development on the Pacific Coast
by Reuben J. Johnson 1

Latest Dredging Practice
by Ole P. Erickson 15

Shoaling of the Lower Hudson River
by Charles M. Duke 29

Marine Oil Terminal for Rio de Janeiro, Brazil
by H. W. Reeves 47

Behavior of Beach Fills in New England
by Harry S. Perdikis 75

Synthetic Material Filters in Coastal Protection
by Hans A. Agerschou 111

Design of Fox Point Hurricane Barrier
by Roy S. Martin 125

(over)

Copyright 1961 by the American Society of Civil Engineers.

DISCUSSION

Page

Great Volga Waterway
by Otakar W. Kabelac 151

Engineering Aspects of Coastal Sediment Movement, by R. Silvester. (September, 1959. Prior discussion: June, 1960. Discussion closed.)
by R. Silvester (Closure) 167

An Electric Analog Model of a Tidal Estuary, by H. A. Einstein and James A. Harder. (September, 1959. Prior discussion: March, 1960. Discussion closed.)
by H. A. Einstein and James A. Harder (Closure) 169

The Supply and Loss of Sand to the Coast, by J. W. Johnson. (September, 1959. Prior discussion: June, 1960. Discussion closed.)
by J. W. Johnson (Closure) 171

Natural By-Passing of Sand at Coastal Inlets, by P. Bruun and F. Gerritsen. (December, 1959. Prior discussion: June, 1960. Discussion closed.)
by P. Bruun and F. Gerritsen (Closure) 173

Breakwaters in the Hawaiian Islands, by Robert Q. Palmer. (June, 1960. Prior discussion: November, 1960. Discussion closed.)
by Charles E. Lee 175
by Robert L. Miller 177

Tetrapods and Other Precast Blocks for Breakwaters, by P. Danel, E. Chapus, and R. Dhaille. (September, 1960. Prior discussion: None. Discussion closed.)
by Shoshichiro Nagai 179
by A. S. Kahan 182

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

SMALL-BOAT HARBOR DEVELOPMENT ON THE PACIFIC COAST

By Reuben J. Johnson,[1] F. ASCE

SYNOPSIS

The scarcity of natural harbor locations and ruggedness of the coastline are primary features effecting engineering and economic analyses of small-boat harbor development along the west coast of the United States. Harbor characteristics, federal interest, benefit analysis, construction features, and shoreline considerations are examined with a view toward ultimate development of a chain of small-boat harbors along the coast.

INTRODUCTION

The Pacific coastline of the four western states of the United States extends some 4,500 miles from Mexico to the Arctic Ocean. A predominant feature of this coast is the close proximity of the mountains to the shore, which results in relatively few favorable harbor locations. This is in contrast with the broad elongated bays and offshore sand islands along the east and gulf coasts which provide many locations for natural harbors. Inlets of major streams, several

Note.—Discussion open until July 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 1, February, 1961.

[1] Chf., Planning and Reports Branch, U. S. Army Engr. Dist., Corps of Engrs., San Francisco, Calif.

bays, and coves or bights in the rocky shoreline provide the principal harbor areas on the west coast. The islands offshore and the sounds in the Alaska area generally have a steep rocky shoreline with few natural harbor possibilities.

Realizing that the shoreline and fishing banks are natural resources that should be utilized for the national welfare, the federal government established a participating harbor development policy in the early part of the last century. A large part of the light-draft harbor development to date on the west coast has been with the Army Corps of Engineers' participation and guidance. To protect and preserve navigation and navigable waters, it is necessary that any public or private agency desiring to construct a harbor in navigable waters first obtain a permit from the Department of the Army through the Corps of Engineers.

SCOPE

The purpose of this paper is to discuss some of the characteristics of light-draft harbors, benefits that may be expected to accrue from these harbors, the general development program from the federal standpoint, and some of the construction problems involved.

DEVELOPMENT INCENTIVES

The first need for harbors-of-refuge and safe mooring areas for light-draft vessels developed as a result of lumber schooner trade and the fishing industry along the coast. With the growth of population, the need for lumber in the cities increased. Since overland coastal transportation was almost non-existent until the 1950's, lumber schooners calling at improvised landings at great risk, carried lumber, minerals, and farm products from these landings along the coast to the growing urban areas. Also, early settlers in this area found the ocean a ready source of income and sustenance from extensive fishing banks. The demand for adequate harbors grew with the lumber and fishing industries. However, in recent years the accelerating interest in recreational boating has emphasized the need for development of additional harbors. Projections based on recent sales of boats indicate that by 1970 at least 75 million Americans will be boating. Translated into boats, this means some 15 million light-draft vessels in the recreational fleet of 1970. The U. S. Coast Guard, as of June, 1958, had registered over 70,000 small craft on the Pacific coast and there are doubtless many more that had not registered at that time. These are all boats 16 ft or more in length and thus could be used in coastal waters. In the past, the lack of harbors has strongly influenced a relatively low per capita boat ownership along the west coast in comparison with the Atlantic and gulf seaboards. As harbor facilities become available on the west coast, it can be expected that the per capita ownership will become more nearly equal.

The trend in size of boats will directly affect the future demand for coastal light-draft harbors. Boat ownership in recent years indicates that the average man starts on a modest scale either with an outboard or a small inboard. He continues to trade-in, or sell and re-purchase, a larger boat every 2 or 3 yr. As the average size of the boats increase, the coastal traffic will become

heavier. The number of casualties resulting from storms at sea and the inability to reach a nearby harbor-of-refuge can be expected to increase if this trend continues.

CHARACTERISTICS OF LIGHT-DRAFT HARBORS

A light-draft harbor by definition is one whose usable depth is limited to 18 ft below mean lower low water (mllw). Also, it should have the following facilities:

1. A safe entrance.
2. A protected anchorage, mooring area, or bulkhead for accommodating transient boats.
3. A channel of sufficient width for maneuvering in and out of the slips.
4. An adequate frontage with piers and slips for permanently based boats.
5. A public landing with provision for sale of motor fuel, lubricants, and potable water.
6. Stalls or slips for transient boats, dockage and repair facilities, telephone service, police and fire protection.
7. Fish handling and storage facilities, where commercial fishing is a justifying feature.
8. Warehouses and loading facilities, where waterborne commerce is used for harbor justification.

The first three items (safe entrance, mooring areas, and maneuvering channel) are required for reasonable accommodation of general navigation and are usually considered a federal responsibility. The remaining items should be provided by the local sponsoring agency.

FEDERAL INTEREST

The federal interest in harbor development along the west coast began with promotion of commerce and provision for defense as initial justifying elements. The Army Corps of Engineers participation in light-draft harbor development has occurred largely within the last three decades with a steadily increasing program up to the present date. The current cost to the federal government on the west coast exceeds $50 million and involves an annual maintenance approaching $1 million. Although the light-draft harbor has some defense value, its primary justification lies in the three basic elements of commerce, fishing, and recreation. The harbor-of-refuge aspect is included in all of these elements.

Prior to the investment of federal funds, each project is carefully studied to ascertain economic feasibility and its need as a harbor-of-refuge. General or nationwide benefits derived from the development of the harbor are distinguished from local benefits or those that will accrue in the local area. Allocation of costs of the project are made on the basis of general and local benefits.

BENEFITS FROM LIGHT-DRAFT HARBORS

The derivation of benefits used in establishing the feasibility of harbor development is considered under four broad categories.

1. Commercial benefits result primarily from reduction in transportation charges. Specific examples are commerce attracted to water movement from a more costly type of transportation, reduction in operating costs of water carriers, or the reduction or elimination of navigational hazards. The general procedure used to develop these benefits includes initially a study of potential development in tributary areas, and extent of use of present navigation facilities to establish the prospective waterborne commerce.

2. Fishing benefits result from elimination of delays in entering and leaving the harbor, spoilage of fish, loss or damage to boats or gear, and a decrease in transportation costs due to proximity of a new harbor to fishing banks. In developing these benefits, the U. S. Fish and Wildlife Service is consulted in regard to extent, type, and sustained yield of fishing banks within the contributing area.

3. Recreational boating benefits are derived by use of a formula adopted by the Corps of Engineers in 1950. The measure of benefit according to this formula is the equivalent to the net return on the depreciated investment in boats received by owners of "for hire" or rental vessels after all expenses have been paid. In arriving at a precentage to apply to the recreational fleet investment as an expression of benefit the following factors are considered:

(a) The estimated net return on "for hire" basis under local conditions.

(b) The percentages for different type craft. These are adopted to reflect the number of persons enjoying the respective craft, with greater weight being given to craft having more per passenger investment. The percentages are applied to the average value of the boat.

(c) The range of percentages. Studies of boating practices in various parts of the country indicate that the net return as previously defined ranges from 6% to 15%. The adopted range of percentage returns by types of vessels is approximately as follows: outboards, 10% to 15%; sailboats, 8% to 12%; inboards, 8% to 12%; cruisers, 6% to 9%; auxiliary sailboats, 6% to 9%.

4. Refuge-harbor benefits are largely intangible, involving primarily the prevention of loss of life and safety in small boat traffic for both commercial and recreational navigation. No attempt is made to assign monetary values to these benefits, but each case is evaluated in terms of its relation to density of small boat traffic, severity and incidence of storms along the coast, and length of coastline without small boat refuge.

Benefits derived from commercial traffic and the fishing industry as a result of the development of the harbor are classed as general benefits. The costs allocated from such benefits are assumed by the federal government. Benefits resulting from land enhancement in the immediate harbor area, and from recreational navigation are considered local benefits. Federal participation in this case is limited to 50% of the cost resulting from allocation of these local benefits.

LOCAL COOPERATION

For each project to be developed by the federal government, a local sponsoring agency is required for certain items of responsibility. These local interests range in size and content from harbor boards or commissions involving the taxable area immediately adjoining the harbor to the entire state in which

the harbor is located. The items of cooperation required for the sponsoring agency generally include the following: (a) provide without cost to the United States all necessary lands, easements and rights-of-way for the construction and maintenance of the project; (b) hold and save the United States free from property damage that may result from the construction or maintenance of the project; (c) provide and maintain without cost to the United States necessary mooring facilities and utilities, including a public landing with suitable supply facilities open to all on equal terms; (d) establish a competent and properly constituted public body empowered to regulate the use and development of harbor facilities; and, (e) make a cash contribution toward the federal first cost of the project when required.

In addition to the foregoing, several other items may be requested of local interests such as: (a) an adequate frontage with stalls or slips for local boats; (b) a public landing with provision for the sale of motor fuels, lubricants and potable water; (c) stalls or slips for transient boats, dockage and repair facilities, telephone service, and police and fire protection; and, (d) facilities and storage for handling of waterborne commerce or fish.

The growing awareness of the need for statewide participation in small boat harbor development in California led to the formation in 1957 of a State Division of Small Craft Harbors and a Small Craft Harbor Commission. Some of the functions of this division include formulating master plans for harbor locations, establishing systems of regulations, assisting in financing, and aiding in local requirements. Funds amounting to $10 million have been provided by a bond issue to implement this program. Loans exceeding $3.5 million have been made to date for small-boat harbor development throughout the state. The other west coast states have not yet made such provision for statewide assistance. Port districts, cities, and counties are entities that assume the local light-draft harbor responsibilities in these states.

The initial action toward investigating the feasibility of a small-boat harbor is a responsibility of the local sponsoring agency. Subsequent to the assignment of such an investigation authorized by the Congress, the Corps of Engineers holds a public hearing in the vicinity of the proposed boat harbor. At that time all information that local interests may have that has a bearing on the project is requested to be presented. A brief description of the type of data that local interests can provide include: (a) a description of the proposed plan or plans that may have been locally developed at that time and the estimated cost thereof; (b) a statement of the necessity for the harbor; (c) a statement of the benefits that may be anticipated from the desired improvements; and (d) the number, type, size, and value of all craft that are expected to use the improvement.

The progress made on federal participation in small-boat harbors is gaged many times by the ability of an organized local group, such as a harbor board, to work directly with the Corps. Such an arrangement provides a mutual understanding and development of those facilities required to be constructed by local people and their relationship to the overall project.

CONSTRUCTION FEATURES

Because of the rugged coastline, most of the harbors on the west coast require structures such as breakwaters or jetties to assure safe navigation. Extensive maintanance dredging in the entrance channels to these harbors is re-

quired because of the large volume of material in littoral transport along the coast. In addition to this dredging, the periodic repair of jetties and breakwaters adds substantially to cost of harbor maintenance. The incessant eroding action of the sea has resulted in damages necessitating redesign and reconstruction of some of these structures several times since initial construction.

Design of breakwater and jetty structures has been largely empirical to date including frequent use of models. The formula now in current use is that developed by R. Y. Hudson, F. ASCE of the U. S. Army Waterways Experiment Station, Vicksburg. Principal factors in this formula for determining the weight of stone required to resist the design wave are side slopes, specific gravity of stone and an empirically developed K factor. The weight of armor stone required to withstand the force of high waves along a coastline directly exposed to the sea ranges from 10 tons to 20 tons on slopes of as flat as 1-on-4. The non-availability of this size of stone in large quantities has resulted in experiments on alternative concrete shapes such as tetrapods and tribars. The use of such shapes has been described elsewhere.

SHORELINE CONSIDERATIONS

The effect of breakwaters and jetties on the adjacent shoreline became a matter of increasing concern in the 1920's and led to the formation by the Chief of Engineers of a Board on Sand Movement and Beach Erosion in January, 1929, and the Beach Erosion Board in July, 1930. Several incidents of serious beach erosion in southern California in the 1930's involving loss of homes and other buildings reemphasized the need for consideration of shoreline changes whenever a structure extends out into the water. It is now required by law to consider, in each navigation study, the effect of breakwaters or jetties on the shoreline for 10 miles on each side of the structure. Sand bypassing plants or other beach replenishment measures are included as a part of the project in the event studies indicate that interruption of littoral drift will become a problem. An illustration of material in littoral transport infringing on the entrance channel and the harbor area is in the Santa Barbara Harbor (Fig. 1).

LOCATION CLASSIFICATION AND EFFECT

Light-draft harbors may be classified in three general locational types: cove or bight harbors, inlet or lagoon harbors, and harbors within natural bays. Each have attendant problems and characteristics as follows:

1. Cove or Bight Harbors. These are formed by a headland or point extending into the sea, or an indentation in the shoreline, which provides natural protection for boats against storms from one or two directions. Under these circumstances an inclosed harbor can be formed by one or two breakwaters. Examples of these harbors are: Halfmoon Bay (Fig. 2) and Redondo Harbor (Fig. 3). Major design considerations include availability of a substantial source of large armor stone, width and location of harbor opening to minimize shoaling of entrance and surge within the harbor, and the effect of breakwaters on adjacent shoreline. The initial cost and maintenance of such harbors are usually the highest of the three types.

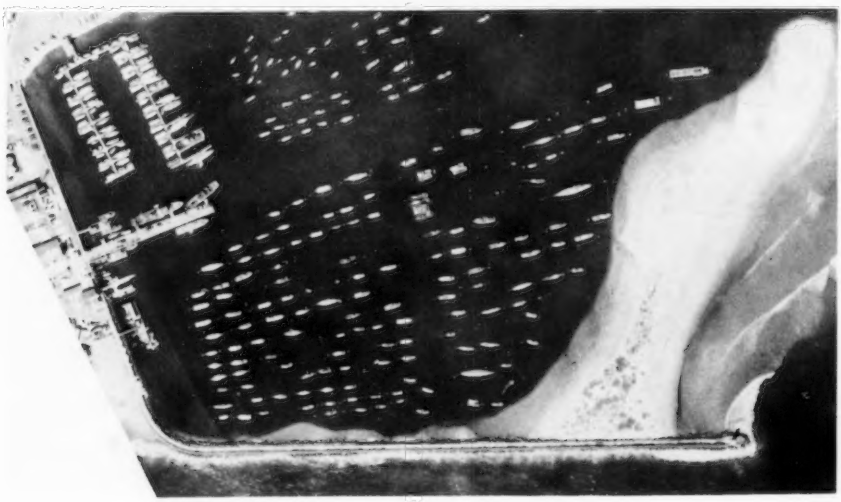


FIG. 1.—SANTA BARBARA HARBOR—SHOALING AT END OF BREAKWATER

FIG. 2.—HALFMOON BAY—NEW BREAKWATERS UNDER CONSTRUCTION

FIG. 3.—REDONDO HARBOR, CALIF. -BREAKWATER EXTENDS INTO OCEAN TO FORM AN ENCLOSED HARBOR

FIG. 4.—MISSION BAY, SAN DIEGO, CALIF.—JETTIES EXTEND THROUGH SAND SPIT. NOTE AQUATIC PARK DEVELOPMENT

FIG. 5.—BODEGA BAY—JETTIES CONSTRUCTED AT NATURAL OPENING IN SAND SPIT TO STABILIZE HARBOR ENTRANCE

FIG. 6.—SAN RAFAEL HARBOR, SAN FRANCISCO BAY—DREDGED CHANNEL THROUGH MUD FLATS TO DEEP WATER IN THE BAY

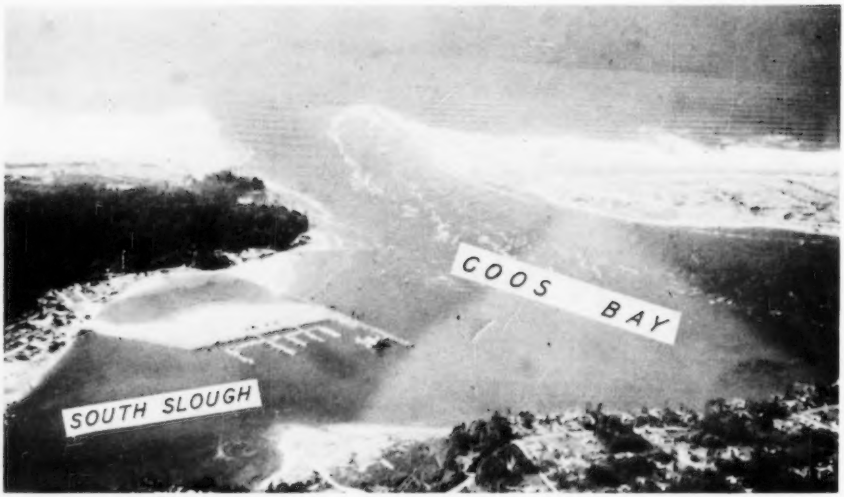


FIG. 7.—CHARLESTON HARBOR, COOS BAY, ORE.—BREAKWATER OFF MAN ENTRANCE CHANNEL

FIG. 8.—BELLINGHAM HARBOR, WASH.—BREAKWATER AND MOORING AREAS

2. Inlet or Lagoon Harbors. These harbor locations are formed by sand deposits at mouths of streams. The sand barrier confines fresh or brackish water in a lagoon during low stream-flow. When flood flows occur, the sand barrier is breached and a natural opening to the sea is formed. A harbor can be developed by stabilizing this entrance channel through the barrier with jetties and rock facing. The littoral drift is thus forced further out into the ocean and precluded from shoaling the natural breach. Sand deposition on one side of the jetties usually occurs while erosion is experienced on the opposite side. The necessity for sand bypassing facilities as a part of the construction of such harbors is a recognized feature. Jetty maintenance and entrance harbor dredging are the major cost items in such harbors. Mission Bay at San Diego (Fig. 4), and Bodega Bay (Fig. 5) are typical illustrations.

3. Harbors Within Natural Bays. San Francisco Bay, California; Coos Bay, Oregon; Strait of Juan De Fuca, Puget Sound and adjoining bays in Washington; and the Inland Passage in Alaska contain numerous light-draft harbors within the protected waters. In some instances the only requirement for such harbors, aside from mooring and docking facilities, is the dredging of an entrance channel and turning basin. San Rafael Harbor in San Francisco Bay is typical (Fig. 6). Other harbors in these relatively quiet waters may require bulkheads or light breakwaters for protection against bay storm waters. Charleston Harbor, Coos Bay, Oregon (Fig. 7) and Bellingham Harbor, Washington (Fig. 8), illustrate this type. Construction and maintenance of harbors within bays are lowest in cost of the three cited. Unfortunately, such large bodies of protected waters are spaced hundreds of miles apart on the west coast.

EXAMPLES OF LIGHT-DRAFT HARBORS

Some of the more prominent light-draft harbors in specific areas along the coast, and pertinent details concerning their development are as follows:

Shilshole Bay, Puget Sound, Washington (Fig. 9).—Capacity, 1,500 boats; cost, $7,760,000, of which $2,460,000 is federal cost; primary benefits accrue to recreation craft with fishing and lumber industries as secondary.

Winchester Bay, Umpqua River, Oregon (Fig. 10).—Breakwater constructed by local interests at a cost of $350,000; federal government dredged channel and turning basin at cost of $54,400. The project was justified (1948) primarily on benefits from savings in operating costs to commercial fishery. In recent years the recreational use has been extensive and is steadily increasing.

Newport Harbor, California (Fig. 11).—Capacity, 4,000 boats; approximate cost, $4,600,000, of which about $1 million is federal; primary use now is for recreational boating. Illustrates ultimate marina development that accompanies harbor development in populous areas.

Marina del Rey Harbor, California (Fig. 12).—Capacity, 8,500 boats; cost $28,020,000, of which $2,320,000 is federal cost. Additional auxiliary improvements by private individuals are estimated at $20 million. Recreational benefits provided the primary justification for this project.

Misson Bay Harbor, California (Fig. 4).—Capacity, 15,000 boats ultimately. Cost, $35 million, of which $10 million is federal cost. Recreation is the primary justifying benefit for this development.

At the present time there are approximately 40 light-draft harbors along the immediate ocean frontage of the west coast of the United States. Between

FIG. 9.—SHILSHOLE BAY HARBOR—BREAKWATER AND MOORING AREAS

FIG. 10.—WINCHESTER BAY, ORE.—BREAKWATER, ENTRANCE CHANNEL, AND BOAT BASIN

FIG. 11.—NEWPORT HARBOR, CALIF.—JETTIES, CHANNELS, AND MARINA DEVELOPMENT

FIG. 12.—MARINA DEL REY HARBOR, CALIF.—JETTIES, ENTRANCE CHANNEL AND BOAT BASIN UNDER CONSTRUCTION

Crescent City and Santa Barbara, a distance of some 570 miles, there are only six harbors-of-refuge which offer all weather protection. The coasts of Oregon and Washington, a distance of some 450 miles, have 16 harbors suitable for small craft. A study by the Corps of Engineers of storm characteristics, small boat capabilities, and storm warning systems indicate that harbors-of-refuge should be available at about 35-mile intervals. It is obvious that attainment of the objective necessary for safe, small-boat navigation is far from realized. However, studies incorporating the foregoing criteria to form a chain of refuge-harbors are currently underway by the Corps. In light of this present day planning, the prospect of future generations safely navigating the length of the west coast in small craft is not beyond reason.

READING REFERENCES

1. "Pacific Coast Recreation Area Survey," U. S. Dept. of the Interior, Natl. Park Service.

2. "Local Participation in Small-Boat Harbor Projects," Waterways Economic Series No. 8, Bd. of Engrs. for Rivers and Harbors, U. S. Army Corps of Engrs.

3. Information furnished by the U. S. Army Dist. Offices in Seattle, Portland, Los Angeles, and San Francisco.

4. "Alternatives to Stone in Breakwater Construction," by Reuben J. Johnson and Olin F. Weymouth, ASCE, Waterways and Harbors Div. Journal, 1956.

5. "Master Plan for California's Small-Craft Harbors," Div. of Small-Craft Harbors, Dept. of Natural Resources, Calif.

6. "Laboratory Investigation of Rubble-Mound Breakwaters," by R. Y. Hudson, ASCE, Waterways and Harbors Div. Journal, March, 1959.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

LATEST DREDGING PRACTICE

By Ole P. Erickson,[1] F. ASCE

SYNOPSIS

A description of the modern hydraulic dredge is presented. Outlined are the latest improvements which have greatly increased production in both soft and hard materials; allowed dredging to greater depths, as well as in exposed waters; and made the hydraulic dredge the most universally used tool for land fills, channel work, and harbor developments.

INTRODUCTION

For more than a century dredges have been deepening rivers and harbors, dredging waterways and canals, making land where no land was before, changing swamps and marshes into prosperous industrial sites or exclusive waterfront subdivisions. There is a fascination about dredging that, once a man has been exposed to it, makes him a life member of the fraternity of dredgemen, none of whom knows all there is to know about dredging, or will ever fully agree with another as to the exact methods of procedure. Possibly within its complexities lies the fascination of dredging.

Hydraulic cutter-type pipeline dredging as known today began its rapid rise about 1875 in San Francisco where, under the direction of such men as Alexis von Schmidt, John McMullen, George W. Catt, R. A. Perry, Claude Cummins, and later around 1900, Stetson G. Hindes, DeWitt D. Barlow and L. T. Gaylord, pioneering dredging companies were formed and expanded throughout the country. They brought deep water to cities, allowing ocean-going ships to sail far inland as on the Houston Ship Canal; they dug thousands

Note.—Discussion open until July 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 1, February, 1961.

[1] Cons. Engr., and Pres., Erickson Engrg. Co., Tampa, Fla.

of miles of inland waterways from Mexico to the Canadian border, reshaped whole sections of shoreline; and they have been one of the greatest factors in the development of the coastal regions of this and other countries.

The important equipment that made possible the modern hydraulic dredge was the invention of the centrifugal pump by Papin in 1705, the steam engine by Watt in 1795, the dredge cutter by Atkinson around 1862, and the ball joint by Robinson about 1900. All these have been improved over the years, and there are now over fifty patents on dredge cutters alone, likewise dredge pumps and ball joints. Machines have been developed which, with various refinements, can now move materials for several miles more economically than in any other manner; they can dig rock, and work in exposed waters subject to ground swells and sea action; and also dredges, with specially designed ladders for digging to 200 ft depths, are now being built.

DREDGE HULLS

Modern dredge hulls, generally made of steel, vary greatly in strength and design. The 10 in. discharge dredges usually have steel hulls with two longitudinal bulkheads, three or more transverse bulkheads. All are oil and watertight with over-all dimensions of 60 ft to 70 ft long, 20 ft to 22 ft wide, and 5 ft to 6 ft deep. The 20 in. dredge hulls average 125 ft to 150 ft long, 32 ft to 38 ft wide, by 7 ft to 11 ft deep with 15 to 20 compartments. The 30-in. dredges are 170 ft to 210 ft long, 45 ft to 50 ft wide, by 12 ft to 14 ft deep, with 20 to 30 compartments. Many dredges in the larger sizes comply with ABS requirements, thus reducing insurance rates. Some large dredges have the stern end of hull and house molded like a ship's bow for ocean towing. However, the barge type rake stern is generally preferred as it simplifies installation of spud well and stern anchor dredging fairleads, while the after end of lower deck house is molded and reinforced to take wave action during ocean towing.

Fig. 1 shows one of the world's most powerful hydraulic dredges. This 30-in. discharge dredge can call on 6000 plus hp on the pump and 1500 plus hp on the cutter. It can dredge to a depth of 60 ft. Fig. 2 shows a 22-in. hydraulic dredge of typical American design. The anchor booms are extended. Fig. 3 indicates a 20-in. heavy duty rock digging dredge of American design.

Sectional hull dredges are now built in sections and bolted together with high strength bolts, using special washers and neoprene gaskets with bolts for water and oil tightness. These can be quickly assembled and disassembled either afloat or ashore for shipment by truck, rail or ship to any location. Before assembly, adjoining surfaces are coated with a non-corrosive compound. The forward A-frame and spud gantries are also sectional and pin connected for easy assembly and disassembly. The strength of the properly designed 14-in. to 30-in. sectional dredge has been proven by extensive rock and hard-material dredging. Sectional hull construction will, on the average, increase the initial cost of a hydraulic dredge about 4%.

The trend is away from crew's quarters on dredges. They are too costly to maintain, increase the size of dredge, and usually dredge crews now prefer

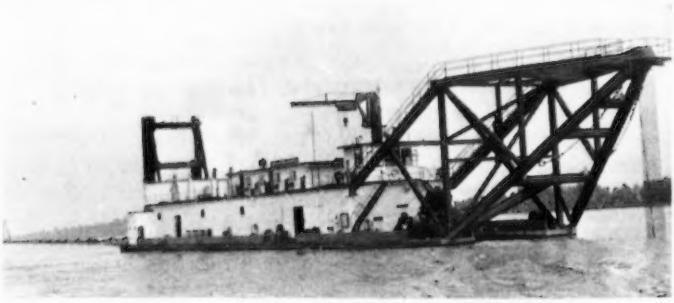


FIG. 1.—30-INCH DISCHARGE DREDGE

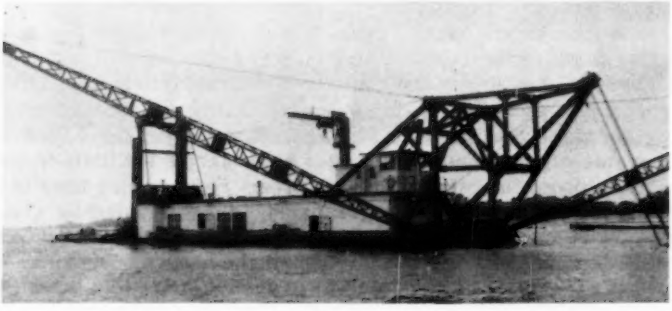


FIG. 2.—22-INCH HYDRAULIC DREDGE

FIG. 3.—20-INCH ROCK DIGGING DREDGE

to live ashore with their families, except in isolated cases where it is cheaper to rent houseboats or build a suitable camp.

DREDGE LADDERS AND FORWARD A-FRAMES

Dredge ladders vary in weight from 15 tons to 250 tons and over, on larger dredges, and from 25 ft to 225 ft in length. They are made from heavy, wide flange beams, built up or tubular, and partially buoyant construction. In heavy materials such as rock dredging, gravel dredging, or both and using long ladders, a special booster on the suction pipe becomes necessary for top production. However dredging may be done to 150 ft with standard ladders in light, silty materials. Many dredges have removable sections for changing ladder lengths for various dredging depths, this is particularly important in rock dredging, shallow dredging, or both. Fig. 4 shows a top view of a direct suction pipe cutter drive ladder for a 24-in. hydraulic dredge. Fig. 5 shows the ladder on the 30-in. hydraulic dredge which is designed for 1500 hp to 2500 hp on the cutter.

Forward A-frames are usually made of steel and are built out over the bow of the dredge to carry the ladder lifting tackle. Experience in rock and hard-material dredging has shown that cable connections between the ladder pick-up tackle and ladder outlast chain, and, what is more important, cable gives warning by showing broken strands before failing.

The A-frame on many modern dredges also carries the built-in structure for the anchor boom backstays. Anchor booms reduce delay time of moving swing anchors and the necessity of having a boat available. The anchor booms are 75 ft to 130 ft long, extending out at 45° from the bow on either side of the dredge. With the anchor booms the dredge operator moves the anchors ahead as required without delays to dredge operation. Where shallow water, or mud flats with little tide variation are encountered, the anchor booms are a vital necessity because they can improve operating time possibly 30%, and greatly reduce the crewmen needed.

SPUD GANTRIES AND DREDGE SPUDS

Spud gantries for lifting the spuds vary from 30 ft to 70 ft or more in height and may be pin connected to the dredge's hull for quick removal. The spud gantries are designed for a quick change over to anchors for rough water, coastwise, or deep water dredging.

Dredge spuds are made completely from steel and vary in diameter from 12-in. extra heavy pipe, or tubing, on 10-in. dredges, to 48-in. OD by 2-1/2-in. wall thickness at top, tapering to 1-1/2-in. at a point on the larger dredges. In some cases they are made from Silicon, T-1, or high alloy cast steel. An adjustable sleeve that can be set 50 ft or more below the hull for dredging with spud sliding inside permits dredging to over 100 ft in depth with spuds. For deeper dredging anchors are preferable to spuds, both as to cost and as to ease of operation.

DREDGE PUMPS

Nearly all new dredges, and some conversions, now use dredge pumps with fabricated or cast outer casings, with all the wearing surfaces lined. Usually

FIG. 4.—DIRECT SUCTION PIPE CUTTER DRIVE LADDER

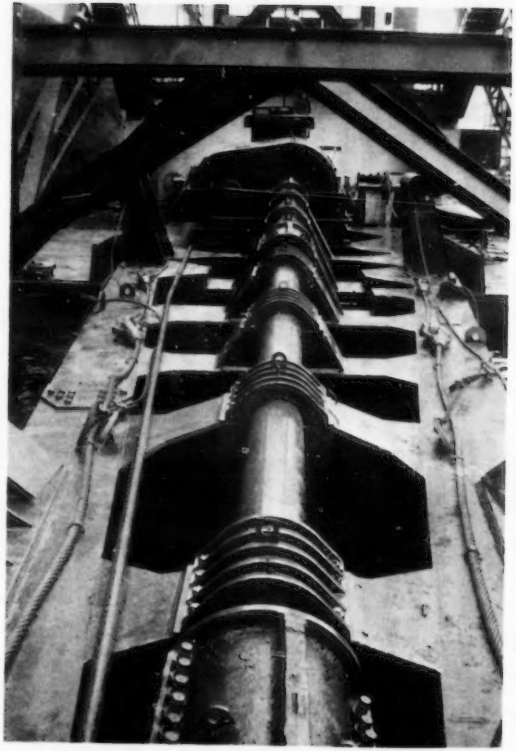


FIG. 5.—LADDER ON 30-INCH HYDRAULIC DREDGE

the liners are of special alloys, 400 to 650 Brinell hardness. The casing liners require little machining, and outwear the old type pump casing two to three times. As the outside casing takes the loading, the casing liner can be practically worn out. This eliminates the danger of worn out casings bursting from high pressure and damaging machinery and sometimes even sinking the dredge. The completely lined pump on large dredges probably reduces pump operating costs 20% or more, depending on the materials handled. On larger dredges average pump efficiency varies from 60% to 75%.

Impellers are generally made of alloy steel for long wear. Large impellers may have the hub cast and the shrouds and vanes fabricated from high-alloy steel and assembled by welding. Impeller speed varies from 900 rpm on 8-in. pumps to 300 rpm on 36-in. pumps, with vane tip speeds up to 9,000 ft per min. Higher speeds cause excessive pump wear. Specific speeds vary from 1,200 ft on 10-in. dredges to 800 ft on 36-in. dredges. Discharge vane angles at tip vary between 20° and 30°, and entrance angles vary 16° to 24°. With few exceptions impellers are now screwed onto the shaft against a shoulder using a double or triple acme thread with filleted corners.

Pump shaft diameters vary from 6 in. to 18 in. with shafts usually made of high alloy SAE 4340, or equal steel to withstand shock loads from boulders and obstructive materials pumped. Kingsbury thrust and radial bearings are used in the United States. Abroad, the Mitchell type is used.

AUTOMATIC RELIEF VALVE

There has become available to dredge contractors an automatic relief valve which reduces "choke-off" from high vacuum and greatly reduces water hammer in pump and pipelines which so often proves costly in high pressure dredging. An increase in production of approximately 5% can be obtained by using this valve. The valve is also beneficial when gaseous materials are pumped, although in extremely gaseous pumping, a gas ejector should also be installed for top efficiency.

HORSEPOWER REQUIREMENTS

Since about 1925 to 1930, horsepower on dredge pumps and cutters has almost doubled. On dredge pumps the horsepower varies from 600 hp on 10-in. discharge dredges to 10,000 hp on 36-in. dredges. Cutter horsepower varies from 200 hp to 400 hp on 10-in. to 14-in. dredges to 1500 hp to 2500 hp on 28-in. and 30-in. dredges and sometimes more.

DREDGE CUTTERS

The cutter is probably the most important item on a dredge. Usually cutters now have three to six blades of built-up construction, with the blades welded to the hub either electrically or by thermit welding. Renewable blade-wear edges are cast in sections, positioned correctly on the blade by bolts and then tack-welded to blades. These sections have various types of teeth cast on them as suited to the material dredged. These are classified as pick points for harder rock, chisel points, spade points or just plain edges for soft rock and for soft material. Recently these edges are bolted on the blades

to reduce the excessive high cost of welding. There are also designs for bolting the blades to cutter hub and backring, eliminating welding altogether.

Cutter speed varies between 12 rpm and 36 rpm depending on the materials pumped and the size of dredge and cutter. D. C. Ward Leonard variable control and A. C. current is used. With A. C. current two, three and four speed motors are used. Motors and cutter reduction gears on most recently designed dredges are placed on the back end of the dredge ladder. Late designs also place adequate thrust bearings in the reduction gear case for taking all thrust loads from cutter operation inside case. This eliminates the expensive outside thrust bearings and their costly and continuous maintenance. Also eliminated on many new dredges with strong ladders are the flexible couplings with their costly maintenance. Fig. 6 shows the ladder and cutter on the 20-in. heavy duty rock digging dredge with 1200 hp on the cutter.

Cutter shaft diameters range from 5 in. to 18 in. and over, with shaft usually made from SAE 4340 steel or equal. Cutter end shaft bearings are usually rubber lined or use laminated phenolic materials. Dredge cutters are now screwed onto shafts against a shoulder, using double or triple acme threads with filleted thread corners. The cutters are quickly removed by reversing the cutter motor or motors.

Wide experience indicates that rock of a Mohs Scale No. 2 and No. 3 hardness can be excavated with hydraulic dredges without blasting. Rock of Mohs Scale No. 4 hardness can also be excavated when blasting is not permitted. Harder rock requires other treatment, such as breaking by dropping chisels, or blasting.

DIRECT SUCTION PIPE CUTTER DRIVE

Since 1956 dredge ladders with direct suction pipe cutter drive have come into use. This eliminates the cutter shaft by attaching the cutter direct to the end of the suction pipe in the usual manner (by threads or wedges). Fig. 7 shows the standard cutter for the direct suction pipe cutter drive. The apparent advantages are about a 20% ladder reduction cost, particularly on large dredges, and the use of smaller, non-clogging cutters. The suction or drive pipe can be made of sufficient thickness to be practically unbreakable. There are now three dredges in operation using this drive, one 20 in., one 22 in. and one 24 in., as well as several smaller dredges.

FORWARD WINCH AND SPUD HOIST

The hauling winch for swinging the dredge in channel on late designed dredges has an average maximum cable pull of about 20,000 lb on 10-in. dredges. Dredges 30 in. or larger have a line pull of 100,000 lb to 130,000 lb at 20 fpm to 30 fpm, and top cable speeds of 175 fpm and over for light dredging. Some dredges have a separate hoist motor for dredge ladder. The spud hoist on most modern dredges is placed at the stern near the spuds. This reduces the spud cable length and wear, and makes for better and cheaper operation. The spud hoist on the majority of new large dredges, or conversions, has three drums so that the dredge can operate on anchors and fairleads (also called "X-mas trees" by old timers) in rough seas or exposed locations, or in over 100 ft deep dredging when spuds cannot be used economically.

FIG. 6.—LADDER AND CUTTER ON 20-INCH HEAVY DUTY ROCK DIGGING DREDGE

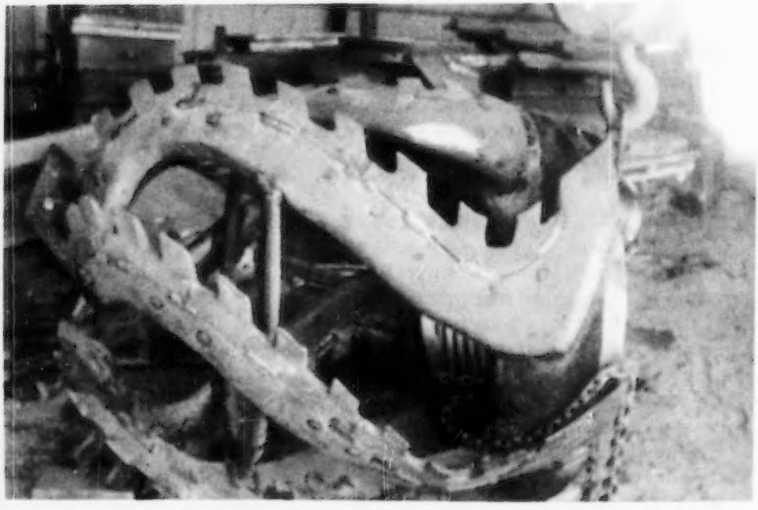


FIG. 7.—STANDARD CUTTER FOR THE DIRECT SUCTION PIPE CUTTER DRIVE

Swing motor horsepower varies from 50 hp on 10-in. dredges to 300 hp or more on 30-in. dredges. Speed control is the Ward Leonard system or Dynamatic control on smaller dredges. Nearly all swing gears are operated by air control or hydraulic system from a central control panel or board in the operating room. So also is the cutter speed variation, and in some cases the dredge pump speed.

GYRO COMPASS AND RECORDING GAGES

Practically all medium to large dredges now have Gyro compass installations with repeaters, width of cut, and some also have depth of cut recorders. They also have recording vacuum and pressure gages. Some have revolutions per minute indicators and recording watt meters on pump, cutter, and swing. Instruments have been developed in Holland which show the velocity and percentage of solids pumped. Fig. 8 gives a partial view of a typical operating room of the latest design. This one is for the 24-in. hydraulic dredge. The records from all these instruments, when analyzed together, pin point the best operators on the dredge and the best method of operating.

POWER PLANTS

Dredge power plants are now divided between diesel, steam turbine, and diesel electric. Dredges up to 26-in. discharge are almost without exception direct diesel driven, sometimes through reduction gears. Above 26-in. steam turbine or diesel electric drives are used for the most part. The trend seems to be to use two large diesels, directly connected to the pump through hydraulic or air couplings and two pinion input reduction gears. The diesel electric dredge will have two to six, or more, high speed diesels driving generators furnishing power for the various motors. On the Pacific coast where shore power is available, there are a few straight electric dredges.

DREDGE HULL AND LADDER PIPING

The ladder and hull suction line leading to the dredge pump varies in ratio somewhat to the discharge pipe according to the individual contractor's point of view. Usually this is in the ratio of 1.27-to-1.60-to-1, or for a 24-in. discharge dredge, the suction pipe would be 27 in. to 30 in. The trend is to the 1.60 ratio or higher with suction mouth belled out 20% to 25% to reduce entrance losses. The suction pipe is usually placed so that the center line of the pipe coincides with the average draft line of the dredge. By having a larger suction pipe the water vacuum carried can usually be kept at 5 in. to 7 in. with 10-ft to 14-ft suction velocity, or 16-ft to 22-ft discharge velocity of mixture pumped. A good dredge pump will generally carry 27-in. top dredging vacuum which, with 6 in. of water vacuum, leaves 21-in. vacuum for material handling. This is most important for top production as it permits the carrying of a high percentage of solids over 40%, with good materials and favorable conditions. With the direct-suction pipe cutter drive the water vacuum is reduced to about 4 in. to 5 in., permitting 22 in. to 23 in. for handling material.

A rubber hose type armored suction connection is now used exclusively between dredge ladder and hull. This allows a much better flow condition to

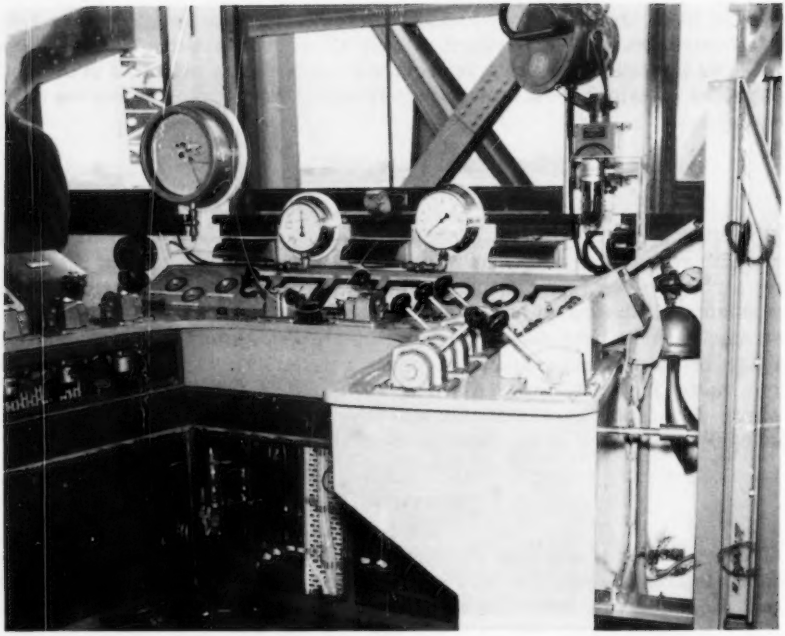


FIG. 8.—TYPICAL OPERATING ROOM

FIG. 9.—TYPICAL DISCHARGE PONTOON LINE

the dredge pump than the old type ball joint or swivel elbow did and requires no servicing except an occasional turning.

The hull discharge pipeline is usually placed on deck outside deck house and connected to the floating pontoon line through a swivel elbow on large dredges. The trend is now away from the swivel elbow to the vertically placed ball joint. This is successful on many large and small dredges.

PONTOON LINE AND BALL JOINTS

The most practical pontoon float has semi-molded rake ends and sides to reduce resistance to current and wave action. The pipe is rigidly fastened by clamps and bolting to a saddle cut down into the top of pontoon, thus lowering the center of gravity of the pontoon and pipe. This makes a cheap, strong and seaworthy pipeline for rough water operation. Fig. 9 shows a typical discharge pontoon line for the 24-in. hydraulic dredge.

Pipeline wall diameters vary in thickness from 3/8 in. to 1 in. and are generally made from abrasive resisting steel. Quite often submerged pipelines are used where heavy traffic is operating; these are sometimes 5,000 ft or more in length. The pipe is coupled together with ball joints, floated to the location, lined up with buoys or piling and sunk. Pontoons made from plastic materials are coming into use on smaller dredges as they greatly reduce maintenance problems.

The application of the flexible ball joint to the floating pipeline permits pipeline pressures of up to 200 lb or higher to be carried on the line, except that for pressures of over 150 lb, a specially designed ball joint is required to avoid the freezing of the joint. Quick-connecting ball joints with wedge type locking glands have been developed and are now (1960) in use. These joints eliminate the use of bolts. Another improved type uses a pinion and rack arrangement which eliminates the handling of the locking gland and can be connected by one man sitting on top of the joint, particularly on smaller dredges.

SHORE PIPELINE Y's AND VALVES

Shore pipe on dredge fills has been standarized to lengths of 16 ft to 20 ft. Thickness of pipe wall varies from 3/16 in. on small dredges to 1/4 in. or 5/16 in. for large dredges, with telescoping joints for average operation. For high pressures, long shore pipelines, or both, flanged joints are used with 1/2-in. to 3/4-in. thickness. This pipe is also of high alloy, wear resisting steel. On shallow fills the shoreline is split up into several lines or branches, connected together with Y's and valves. These are usually power operated, either hydraulically or by tractor. The trend is now to mechanize and reduce labor costs. Light crawler cranes and tractors are standard equipment for handling pipelines. Constant radio contact between dredge and shore pipelines is general practice by using a loud speaker or squawk box on shore.

PIPELINE VELOCITIES

Pipeline discharge velocities vary greatly according to individual contractors. Generally for dredges with 8-in. to 12-in. discharges velocities varying from 10 fps to 18 fps have proven satisfactory depending on type of

material, length of line to pump, and so on. On the larger dredges, with 16-in. to 30-in. discharges, velocities from 14 fps to 26 fps and even higher are carried.

There are several formulas for computing friction in pipelines. Such as those of Scobey, William and Hazen, Darcy and many others. Of these, the writer prefers the Scobey formula as it is simple and can be committed to memory and solved by slide rule. However all formulas require assumptions for specific gravity of the materials handled, the percentage of solids pumped, and the straightness of pipelines. Unless reasonably correct allowances are made, these computations can be misleading.

It is seldom that top production can be obtained in average materials with velocities over 18 fps due to increased pump cavitation which lowers top digging vacuum. As for instance a 24-in. dredge at 18 fps velocity in soft materials may carry up to 40% solids, or pump 3,000 cu yd per hr, whereas at 24 fps velocity only 28% solids, or about 2,800 cu yd per hr, can be obtained. Too high velocities increase horsepower requirements as well as increase pump and pipeline wear. Therefore, serious consideration should be given to velocities in reference to production.

BASE OF OPERATIONS

The majority of all large dredges have a base of operations set up adjacent to the project on shore, or on a large covered barge nearby. The barge has the advantage of being easily moved along with the dredge and also dredge equipment on it can be stored when moving. On this base or barge, rather complete shop facilities are set up for equipment repairs, especially for welding and minor machine work. If rock is being dredged, a large welding force is necessary to keep the dredge cutter teeth sharpened.

DREDGE CREWS

Operating personnel on dredges vary with the size of dredge and project conditions. Usually on 10-in. dredges, there is a total crew for 24 hr of fifteen to eighteen men. The 20-in. dredges have about forty to fifty crewmen. The 30-in. dredges use approximately seventy-five to one hundred men.

PRODUCTION

A well designed 10-in. dredge, with 400 hp to 600 hp on pump and 75 hp to 200 hp on cutter, on 2,000 ft of pipeline, will conservatively pump 250 to 400 cu yd per hr in soft materials and 50 to 100 cu yd per hr in soft rock.

A good 20-in. dredge, with 2000 hp to 2500 hp on pump and 750 hp on cutter, will pump 1,200 to 2,500 cu yd per hr in soft materials and 100 to 800 cu yd per hr in soft to medium hard rock on pipelines up to 10,000 ft. A well-designed 30-in. dredge, with 5000 hp to 8000 hp on pump and 1500 hp to 2500 hp on cutter will pump 2,000 to 4,500 cu yd per hr in soft materials, and 200

to 2,000 plus cu yd per hr in soft to medium hard rock, and on pipeline lengths up to 15,000 ft and longer.

OPERATING COSTS

The average over-all operating cost for 10-in. dredges, for 24 hr is $15,000 monthly. For 20-in. dredges approximately $60,000 monthly. And for 30-in. dredges $100,000 and over. These figures vary depending on the difficulty of the project and the resourcefulness of the contractor. Under average dredging conditions 22 operating hr a day are obtained and quite often 23 hr. In rock and abrasive materials this performance drops to 18 or 20 hr to necessary renewals of cutter teeth, pump and pipe wear, and other associated problems.

INITIAL CONSTRUCTION COST

Initial cost of well-designed modern hydraulic dredges range from $225 to $275 per hp. The direct diesel drive dredge is the cheapest dredge to build, followed by the steam turbine and the diesel electric. While the diesel electric dredge is the most expensive, it is also the most flexible in operation due to ease of power control. The direct diesel drive dredge is by far the most popular and the best money maker. There are more of this type of hydraulic dredges being used than all the other types combined.

CONCLUSIONS

The trend is to higher and higher horsepower on the main pump and cutter. All lined dredge pumps are coming into use, for they can probably be operated 20% cheaper, particularly in abrasive materials. Pump and cutter shafts are now generally made of high-alloy steel. Dredge ladders are now of sufficient strength so that deflection from dredging hard materials is less in the ladder than in the cutter shafting. This reduces shaft failures and the need for flexible couplings on shafting. Dredge spuds are larger in diameter and they are of structural or cast-alloy steel. Dredge cutters are now usually built with removable wear edges, cast in two or three sections, with various types of high-alloy steel teeth cast directly on the edges. They are positioned and tack welded to blades or bolted directly to the blades with high-alloy steel bolts.

Anchor handling booms that heretofore were used on few dredges are now (1960) in almost universal use, and they will, under adverse operating conditions, reduce downtime by several hours per day.

Probably the most important new development is the direct suction pipe-cutter drive which combines the function of the cutter shaft and the suction pipe and eliminates the costly cutter shaft and its maintenance.

Pontoon line ball joints that eliminate the usual bolted connections are coming into use, permitting the line to lay lower in the water and making it more stable and less effected by wave action. Pontoons can be smaller, and the connecting time for the ball joints is cut almost in half.

Additional research should be made on the development of abrasive resisting materials for use in dredge pumps, liners, cutter teeth and blades, and dredge pipe for the reduction of maintenance costs and reducing the downtime of the dredge. Also, there should be more efficient shore fill pipeline handling equipment developed for soft material operations.

The hydraulic dredge is becoming the most commonly used tool for both hard and soft material handling in subaqueous excavation. Not only can it pump onto shore fills, but it can be used in restricted locations, where spoil areas are not available, by pumping the material into barges, thus taking the place of the dipper, bucket dredges, or both. Dredges with 36-in. discharge are now in operation. Preliminary design of dredges for special projects with up to 60-in. discharge have been made. These will be capable of pumping several miles with estimated capacities of 10,000 cu yd per hr in soft materials and 3,000 to 4,000 cu yd per hr in rock of No. 2 and No. 3 Mohs Scale hardness.

All this is a far cry from the first hydraulic dredge made by Hoffman in about 1850 in Germany for dredging canals and filling marshes (with a production rate of about 30 cu yd per hr) and from the primitive equipment used by the Egyptians in the original building of the Suez Canal some 4,000 yr ago.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

SHOALING OF THE LOWER HUDSON RIVER

By Charles M. Duke,[1] F. ASCE

SYNOPSIS

A major item of maintenance, and a deterrent to the full use of the Hudson River frontage for maritime purposes, is the heavy shoaling which occurs in the channel and adjacent slips of the lower estuary. There is discussed herein the progress of investigations to find a solution to this problem. These include the sources of sedimentation, the characteristics of the fresh and salt water components of flow within the estuary, the quantity of shoaling which occurs, and the fundamental principles affecting the sedimentation process. Because of the non-analytic character of the complex factors involved in this problem, two hydraulic models are being used to prove a number of possible solutions. The first model reproduces the entire New York Harbor, and the Hudson River up to the normal limit of salt-water intrusion at Poughkeepsie, N. Y. (a distance of 100 miles), has a horizontal scale of 1:100 and a vertical scale of 1:100. A second enlarged model reproducing the lower Hudson River from about 1 mile upstream of the Battery to about 1 mile downstream of the George Washginton Bridge (a distance of about 9 miles) has a horizontal scale of 1:300 and a vertical scale of 1:100. This model is designed to study the local slip shoaling. Although model studies and analysis of results have not yet reached a point where it can be said that a final solution has been found to the problem (as of 1960), interim conclusions are presented on the results of some twenty plans and variations tested. These include the use of sedi-

Note.—Discussion open until July 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 1, February, 1961.

[1] Col., Dist. Engr., U. S. Army Engr. Dist., New York, N. Y.

mentation basins, the realinement of the deep-water channel by fills and dikes, and the closure of flow from one of the several component waterways.

INTRODUCTION

The lower Hudson River is the principal artery for the movement of maritime commerce to the Port of New York, to upriver ports including Albany, and to the hinterland beyond. Heavy shoaling in the lower river is currently (1960) preventing full use of the river frontage for maritime purposes. This paper is concerned with the studies currently being conducted by the Corps of Engineers to solve this shoaling problem.

FIG. 1.—BASIN LOCATION

For proper perspective, it should be pointed out that this problem is typical, because all fresh-water streams discharging into tidal salt waters have similar shoaling problems involving flocculation and settlement of fine materials in suspension. It is exceptionally complex, however, because the interplay of alluvial and tidal currents in the lower Hudson defies mathematical analysis. It is also of great magnitude, because more than 4,000,000 cu yd of material

are deposited annually and then removed at a cost of about $5,000,000 in a major channel of the world's busiest port.

BASIN CHARACTERISTICS

The Hudson River basin (Fig. 1) covers 13,400 sq miles, mostly in northerly and easterly New York State, with small areas in New Jersey, Connecticut, Massachusetts, and Vermont.

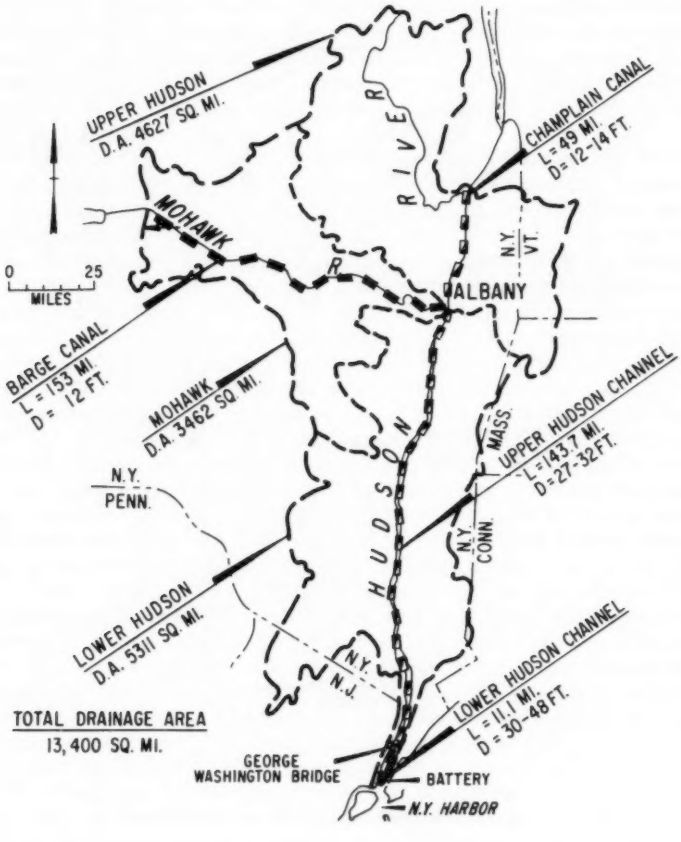


FIG. 2.—BASIN CHARACTERISTICS

The basin is roughly cross-shaped (Fig. 2), extending 120 miles east to west and 236 miles north to south. From its source in the Adirondack Mountains, the river flows south to Albany where it is joined by the Mohawk, its

principal tributary. The main stream then continues for 145 miles to its mouth at the Battery, New York City, where it enters Upper New York Bay. Below Albany the river is tidal with a mean range of 4.4 ft.

The major physiographic features of the watershed (Fig. 3) are a mountainous terrain covering 43% of the basin (slight erosion); cultivated lands covering 47% (moderate to severe erosion); and urban developments over 8% (slight

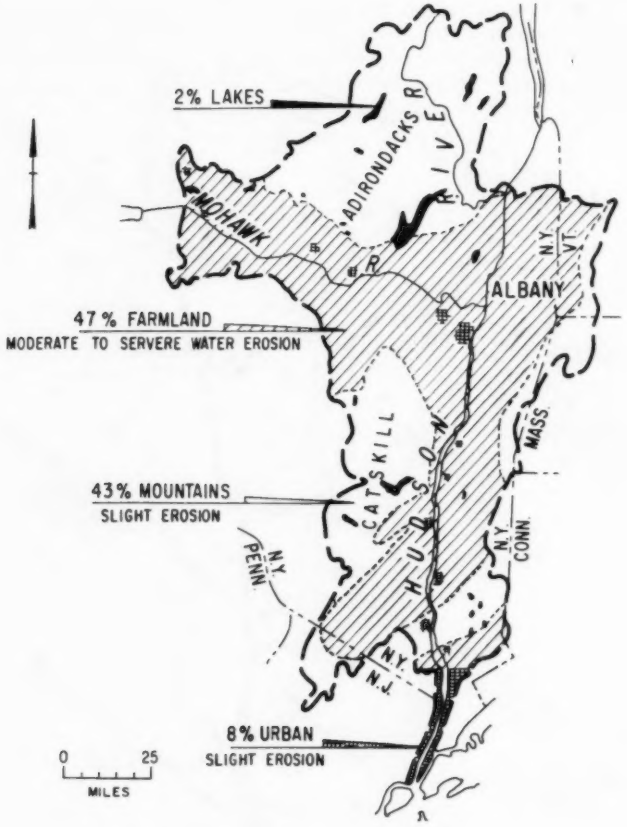


FIG. 3.—PHYSIOGRAPHY AND EROSION

erosion). Lakes and water bodies cover the remaining 2% of the basin. Hydrologically, the average annual precipitation is 43 in., and the annual run-off at the mouth is about 24 in. This corresponds to an average fresh water run-off at the mouth of about 23,000 cfs.

At its mouth, the Hudson has been improved for navigation to provide depths varying from 30 ft to 48 ft. Above this to Albany the nabigable depth is 27 ft

and work is underway to make it 32 ft. North of Albany, the river has been canalized with depths of 12 ft and 14 ft, connecting with the narrows of Lake Champlain. West of Albany the Mohawk has been canalized to a depth of 12 ft, forming part of the New York State Barge canal system connecting ultimately with Lakes Erie and Ontario of the Great Lakes system.

FLOW CHARACTERISTICS

From the foregoing discussion, it is apparent that there is an ample source of shoaling material which can be transported into the lower problem area. With this in mind, attention should now be directed to the complex circulation of fresh and salt water in the lower River (Fig. 4), which has a great influence on the transportation and deposition of shoaling material. (The general circulation patterns in the New York Harbor were described in some detail by E. P. Fortson and H. B. Simmons in a paper presented at the October 1959 ASCE Convention in Washington, D.C.)

The principal hydraulic components are Lower New York Bay, the Narrows, Upper New York Bay, Sandy Hook Bay, Raritan Bay, Newark Bay, Arthur Kill and Kill Van Kull, Hudson River, East River, Harlem River, the lower reaches of the Hackensack and Passaic Rivers, the Atlantic Ocean, and Long Island Sound. The tidal ranges throughout the system are influenced by the Ocean, the Sound, the Hudson River, and the lengths and cross sections of the interconnecting channels. Under average conditions of tide and fresh water discharge, the tidal currents in New York Harbor are relatively weak. Maximum ebb and flood currents for spring tides are shown on this chart.

The peculiarities of the tidal circulation in New York Harbor can be demonstrated by analyzing the current pattern 2 hr. after low water at the Battery (the lower tip of Manhattan). At this stage of tide the ocean flood current is well in Lower New York Bay. The Hudson River is still ebbing at the surface although the flood tide has commenced on the bottom. Newark Bay and the East River are receiving ebb discharge from the Hudson River on the surface and flood discharge from the Upper Bay at the bottom. At the same time, the Upper Bay is receiving ebb discharge from the Hudson and flood flow from the Narrows. Thus, Upper New York Bay discharges into two tributaries and receives discharge from two others simultaneously. This difference in phasing of the currents in the various components of the harbor exert a definite influence on the establishment of shoaling patterns in the lower Hudson River.

As a result of this complex interaction of fresh and salt waters, the harbor south of the Battery, the Kills, and the East and Harlem Rivers may be classified as a well-mixed estuary, because tidal flood discharges predominate so greatly over fresh-water flows that, even for the weakest currents, the vertical mixing of salt and fresh water is virtually complete.

That portion of the Hudson from the Battery to Haverstraw Bay, well upstream of the George Washington Bridge, may be classified as a partly mixed estuary, since the vertical mixing of the salt and fresh water is far from complete. In this area, salinities at the bottom may be as much as 10 parts per thousand greater than at the surface.

A study of current-velocity charts developed at hourly intervals (Fig. 5) between the times of high and low water was made for the problem area between the Battery and the George Washington Bridge. This study revealed that currents favorable to the sedimentation process, or velocities less than 2 fps, ex-

ist during an average tide for a period of 1 hr before and after slack water, or a total of 4 hr in each tidal cycle. During the remainder of the cycle, currents are strong enough to prevent deposition, but are still not of sufficient strength to sweep the sediment to sea.

Because of the vertical-density gradient, the reversal of tidal current from ebb to flood occurs as much as 2 hr earlier at the bottom than at the surface.

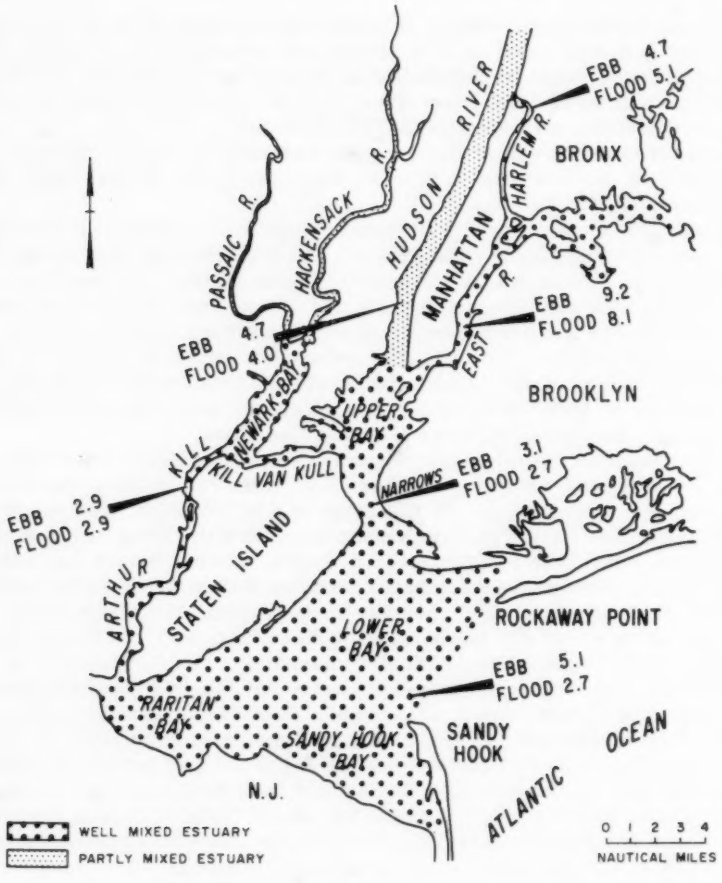


FIG. 4.—MAXIMUM EBB AND FLOOD CURRENTS IN FEET PER SECOND

Since the strengths and durations of the bottom flood currents are greateer than those of the bottom ebb currents, more water flows upstream than downstream at the bottom during a complete tidal cycle. Conversely, more water flows downstream than upstream at the surface. The length of the river over which the bottom currents are predominantly upstream, as well as the position in the

vertical below which the upstream current predominate, is naturally affected by the volume of fresh-water run-off of the Hudson.

In Fig. 6 are shown the results of a computation of flow predominance in the vertical at a station located in the center of the Hudson River opposite 111th Street. Here it can be seen that upstream flow predominates over downstream flow below the 55%-of-depth level.

Fig. 7 shows the same vertical relation as in Fig. 6, related to distance above the mouth of the river going from right to left. The dotted lines show the condition for a fresh-water run-off of 6,000 cfs from the Hudson, with the comparable condition under a 36,137 cfs run-off superimposed by solid lines. The significant point to be derived from Fig. 7 is that the predominance of

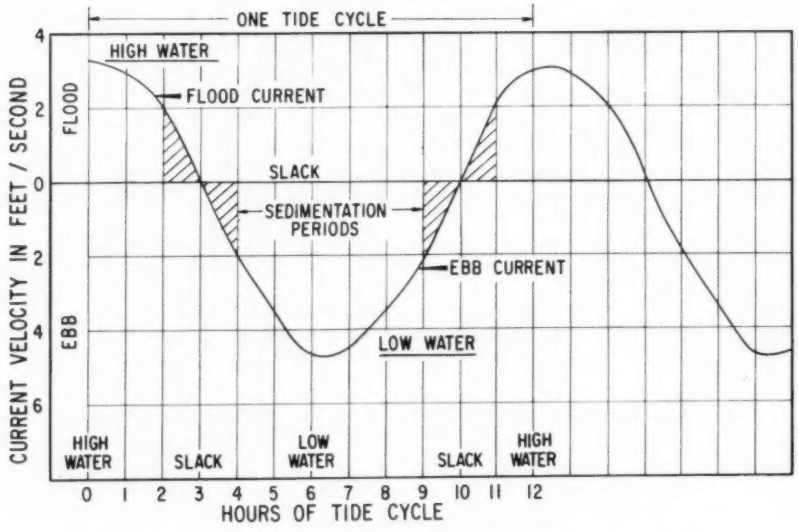


FIG. 5.—AVERAGE TIDAL CYCLE-WEST 23rd TO 129th ST., N. Y.

flow between the Battery and the George Washington Bridge is upstream, regardless of the magnitude of fresh-water runoff. This is primarily attributable to the fact that, right at the Bridge, the cross-sectional area of the River is only about 60% of the area immediately adjacent on both sides.

This local disruption is an important factor in the shoaling problem for two reasons. First, the vertical mixing of salt and fresh water is enhanced because of the higher tidal current velocities in this restricted section. Hence, with a sudden increase in cross section at a point where the bottom flood predominance is enfeebled, sediment which might otherwise be induced to proceed further upstream is deposited.

Secondly, the Harlem River joins the Hudson just upstream of the bridge and during certain phases of the tidal current, water with a relatively high saline content is discharged into the Hudson from this source. This tends to

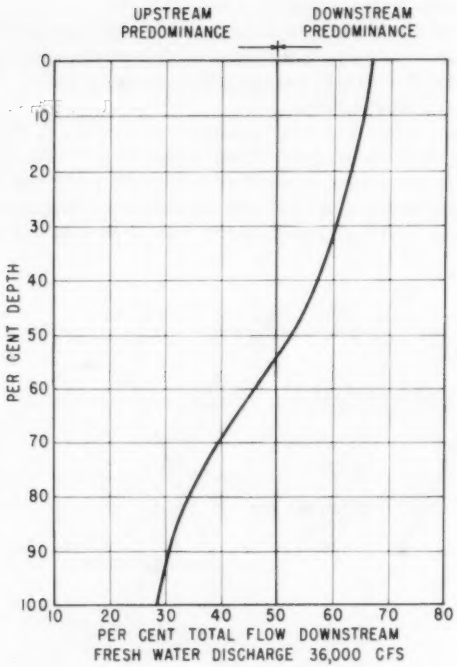


FIG. 6.—VERTICAL DISTRIBUTION OF FLOW DURING TIDAL CYCLE

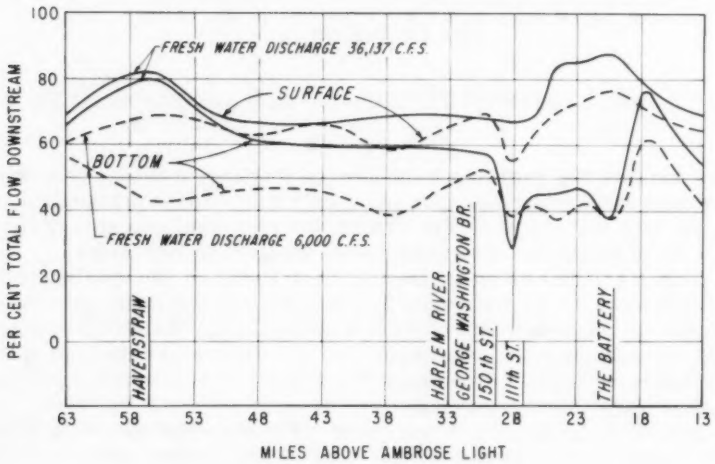


FIG. 7.—FLOOD PREDOMINANCE-SURFACE AND BOTTOM

disrupt the vertical circulation pattern in the Hudson at this point and to contribute significantly to the conditions favorable to sediment deposition.

THE SHOALING PICTURE

The principal channel in the Hudson below the George Washington Bridge is 45 ft deep for the middle 2,000 ft and 40 ft deep for the remaining width from the Battery to 40th St. Above this, it is 48 ft deep for the middle 2,000 ft and

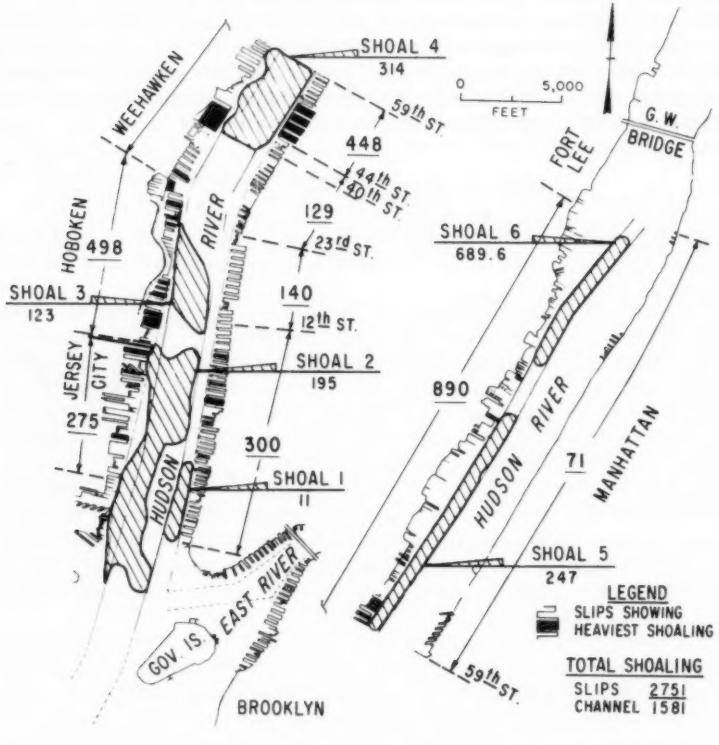


FIG. 8.—SLIP AND CHANNEL SHOALING IN THOUSANDS OF CUBIC YARDS

40 ft deep for the remaining width to 59th St., where it drops to 30 ft deep and 750 ft wide along the New Jersey (west) shore, with a natural deep water channel 48 ft deep on the New York (east) side.

The shoaling in the Federal channel is not uniform. It concentrates in six major channel areas and the slip piers immediately adjacent. The total shoaling in these six areas averages 1,581,300 cu yd annually, divided among the areas shown in Fig. 8. It can be seen that the heaviest shoaling, almost

1,000,000 cu yd per yr, is in the Federal channel along the Weehawken-Edgewater shore in Areas 5 and 6.

The shoaling in the pier slips, as in the Federal channel, is not uniform. Shoaling averages 2,750,000 cu yd annually in the areas shown in Fig. 8. The heaviest shoaling, averaging about 900,000 yd annually, takes place in the slips adjacent to Shoal Areas 4 and 5, in Weehawken and Edgewater, N. J.

The development along the lower Hudson River has been intense. Industries have established plants on the waterfront, attracted, in part, by the economic advantage afforded by deep-draft water transportation at their doorstep. However, the burden of slip dredging has become too great for many of them. As a result, some have moved elsewhere, leaving their plants unoccupied, while others have accepted the inefficient rehandling of cargoes by means of shallow-draft lighters as a substitute for the more costly dredging. Only three firms along the Edgewater waterfront are presently maintaining their slips for deep-draft vessels, while still others, although still in business, have abandoned their waterborne commerce completely.

CONCEPTS ON SEDIMENTATION

With the shoaling problem now defined, it should prove helpful to divert briefly and examine some of the fundamental concepts governing the internal sedimentation process.

The sediments being deposited in the Hudson estuary may be classified as "primary" and "secondary." By way of definition, primary sediments are those deposited directly from water entering the estuary, whereas secondary sediments are redistributed within the tidal section by local currents. The principal source of sediment is, of course, the upper watershed, but eroding stream banks, the adjacent ocean area, and industrial wastes and sewage all contribute to the availability of material.

From preliminary results of a 3-month sampling program at Poughkeepsie, it is estimated that about 3,200,000 cu yd, representing 76% of the material deposited in the shoals annually, originates directly from the watershed. The remaining 24% can be attributed to secondary sources, including redistribution of bottom materials by tidal currents and agitation during dredging or construction operations.

The movement of material in a tidal waterway and its ultimate deposition in a shoal is not the simple process of a single deposit in a given location, but the end result of a series of movements. In the upper reaches of the river, where the fluvial characteristics predominate over the tidal, materials carried as bed load or in suspension settle when the transporting power of the current weakens. When the loss of this transporting power is gradual, the materials are sorted by size and distributed over a considerable area. Similarly, when the velocity change is abrupt, the materials are deposited within a short distance of the change in regimen. Under tidal influence, the heavier particles are deposited in the upper reaches of the estuary although the lighter particles, though perhaps deposited momentarily during periods of slack current, are again placed in suspension and transported farther downstream during successive current cycles before finally being deposited in areas where current velocities under normal tidal conditions are too weak to generate further movement.

In addition to those particles carried as bed load or in suspension are many fine colloidal particles which are kept in suspension by Brownian, or molecular, movements. When these colloidal particles make contact with salt water in the estuary, they are activated electrolytically to combine in oppositely charged pairs to form a colloidal gel. This process, known as flocculation, continues as the material moves downstream. The generally recognized principals effecting this process are as follows:

1. Flocculation will be most active where the fresh water first mingles with salt water near the interface between salt and fresh water, and will diminish downstream as the amount of unflocculated material progressively decreases.
2. Flocculated material will settle to the bottom through the salt-water wedge most rapidly where the wedge is either stagnant or has low velocity and little turbulence. The deposition will therefore be governed by the behavior of the salt-water wedge, and the material will be carried alternatively upstream and downstream as the wedge advances and retreats with the tides. Since this wedge is on the bottom of the river, the predominant movement will be upstream, as pointed out previously.
3. After the material reaches the bottom, it compacts gradually into a sticky deposit commonly called "marine clay" which resists subsequent removal by currents more effectively than sand, shell, or non-colloidal silts.

The amount of flocculation is affected principally by the salt-water to fresh-water ratio (10% to 15%) and to a lesser degree by the reactive components of the soil, its pH value, temperature, and organic composition.

Virtually all of the soils particles involved in the flocculation process are initially of 0.002 mm size or smaller; in fact, particles over 0.074 mm in diameter hardly participate at all. It is also characteristic of this process that the water must be acid, as alkaline water tends to reverse the process.

Other factors favorable for flocculation are: (a) A pH of less than 7, (b) high water temperatures (above 30 °F), (c) the presence of sewage and industrial concentrations over 3.5%, and (d) turbulence.

An analysis of a typical water and silt sample from the problem area is shown in Table 1 and indicates many conditions favorable to flocculation. The character of the shoaling material is shown in Table 2. Downstream samples from the 40-ft channel averaged 16% sand, 73% silt, and 11% clay. Further upstream, in the 30-ft channel, the samples averaged 7% sand, 78% silt, and 15% clay. Furthermore, the bulk of the shoaling consists of a soft colloidal surface stratum underlain with firmer compacted material. This is illustrated by the spread noted in Table 2 for specific gravities (2.6 to 2.8), water contents (54% to 206%), and wet in-place densities (1252 to 1649 g per l). The material composition of this sediment expressed as a percentage of total dry sample weight was 83% of acid insoluble matter, of which the bulk, (61% percent) was silica.

During a tidal cycle, the face of the salt-water wedge moves upstream and downstream over a distance of several miles. Although the currents in both the fresh water and salt water strata usually reverse with tidal phase, the downstream currents predominate in the fresh-water stratum, whereas the upstream currents predominate in the salt-water stratum. The maximum currents in the fresh water are usually near the surface and at the estuary entrance, while those in the salt-water stratum are usually near the bottom and

TABLE 1.—FACTORS INFLUENCING FLOCCULATION AND SEDIMENTATION PROCESS

Factor	Average Water and Sediment Sample 30 ft to 40 ft Channel	Conditions Favorable for Flocculation
Salinity	43%	More than 10% to 15%
Particle size	80% range, 0.006 to 0.074 mm	Less than 0.074 mm
Soil component	Acid	Acid
pH	6.5	Less than 7
Temperature	30°F to 79°F	More than 30°F
Sewage and industrial waste	3% to 27%	High
Turbulence	High	High
Organic matter	8%	Variable

TABLE 2.—ANALYSIS OF SHOALING MATERIAL IN 30 FT AND 40 FT CHANNEL

Specific gravity	2.60 to 2.79
Water content	53.6% to 205.5%
In place wet density	1252 to 1649 g per l
Loss on ignition	10%
Acid insolubles	83%
Silica	61%
Alumina	13%
Iron oxide	6%
Titanium, calcium, and magnesium oxides	3%
Organics	8%
Other materials	9%

at the upstream limit of salt-water intrusion. The regions of heaviest shoaling lie between the high tide and low tide positions of the saline interface.

Shoaling in the navigation channel of the Hudson occurs between the George Washington Bridge and the Battery, particularly in the 30-ft project channel which serves the Weehawken-Edgewater area. Light-weight sediments, which enter the bottom strata because of flocculation or other reasons, are moved progressively upstream in the naturally deep channel (48 ft to 50 ft) along the Manhattan side of the river until they reach the vicinity of the bridge. Here upstream progress stops due to the disruption of the predominance of bottom flow caused by the constriction in the cross-sectional area previously noted. As a result of the intense vertical mixing at this point, the sediments are resuspended in a state subject to quick redeposition. The downstream flow predominance of the upper strata of the river then moves the material to the problem shoal areas in the shallow channel along the New Jersey shore.

Condition surveys substantiate the foregoing analysis by demonstrating that the shoal which forms in the 30-ft channel always begins at the upstream end and progresses downstream. Except for the vertically-mixed region at the bridge, it is probable that sediments being moved upstream in the bottom strata would be transported well above the bridge before being deposited; in such event, it seems logical that shoaling of the 30-ft project channel would be appreciably reduced.

Shoaling in the navigation slips is attributed primarily to local current action. The average entrance current velocities during a tidal movement are too low to account for the rates and distribution of shoaling observed in the slips. Because the slips are generally oriented at right angles to the channel, there is a strong tendency for the higher-velocity currents to flow past the slip entrances, except for the slower-moving water on the bottom, which enters the slip. The inflowing water is largely confined to a 3 ft to 5 ft stratum on the bottom, where the concentration of sediment is highest and where the local velocity is sufficient to transport the sediments over the full length of the slip. During the falling phase of the tide, the slip discharges uniformly over its entire depth and width. Thus, the sediments which enter the slip in the concentrated inflow adjacent to the bottom are not flushed out by the uniform outflow during the emptying phase.

SOLUTION TO THE PROBLEM

It is evident that the phenomenum of shoaling in a tidal estuary is not simple. The complex interrelation of forces and influences do not lend themselves to analytical solution. In order to develop a solution to the Hudson River problem, the Corps of Engineers decided to resort to the use of hydraulic models. A model study is now (1960) underway at the U.S. Army Engineer Waterway Experiment Station at Vicksburg, Miss.

The prototype area was reproduced in one model (Fig. 9) with a horizontal scale of 1:1000 and a vertical scale of 1:100. This model includes the entire New York Harbor and the Hudson River up to the normal limit of salt-water intrusion at Poughkeepsie. It extends far beyond the immediate problem area to insure that all factors such as tides, currents, and salinity are accurately reproduced, and that any plan to alleviate the existing shoaling problem does not create a future problem in other areas of the harbor.

After a series of studies with the comprehensive model, it was decided that a supplemental model, to a larger scale, would be required to study the pier-slip shoaling. The pier-slip model extends from 1 mile above the Battery to about 1 mile below the George Washington Bridge. The vertical scale is the same as the comprehenisve model, 1:100, but the horizontal scale is 3-1/3 times as large, or 1:300.

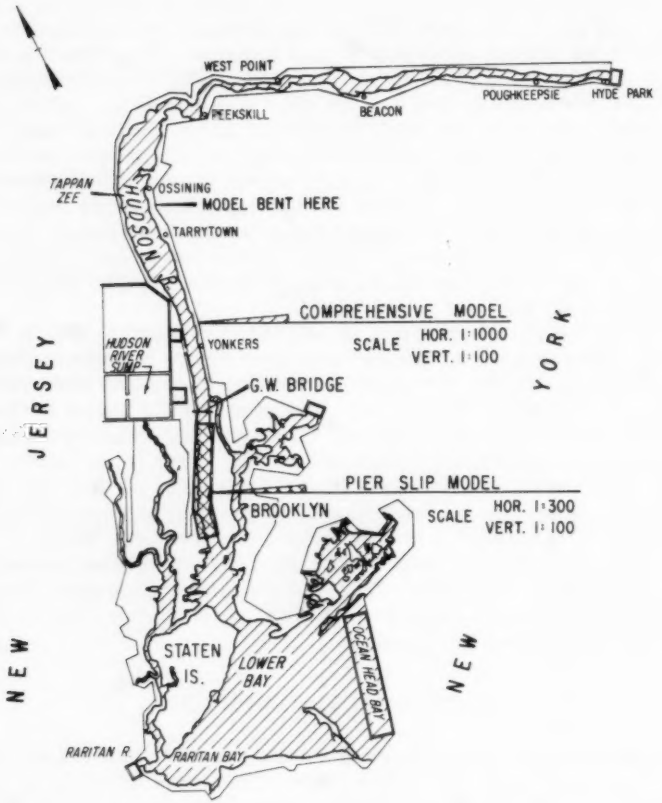


FIG. 9.—MODEL LAYOUTS

At this time (1960), the model studies and analyses of results have not reached a point where a positive plan for corrective action can be recommended. Some interesting observations can be presented from the studies conducted to date, however. About twenty plans, including variations, were tested, such as the provision of sedimentation basins in the existing navigable channel, realinement of the deep-water channel by construction of dikes, modification of the channel geometry upstream and downstream of the George Washington Bridge, and a closure of the Harlem River.

In any solution there are two basic—and often conflicting—considerations, one involving hydraulic factors and the other, economic implications. These two considerations are not necessarily satisfied fully by any solution and, as in many such matters, a compromise area must be found.

Fig. 10 shows two different schemes tested in the model, one utilizing sediment basins and the other, a channel-constricting wing dike.

Of the sediment basins tested, the three shown proved to be most promising. These basins are 6,000 ft to 7,000 ft long and 50 ft to 60 ft deep, with a

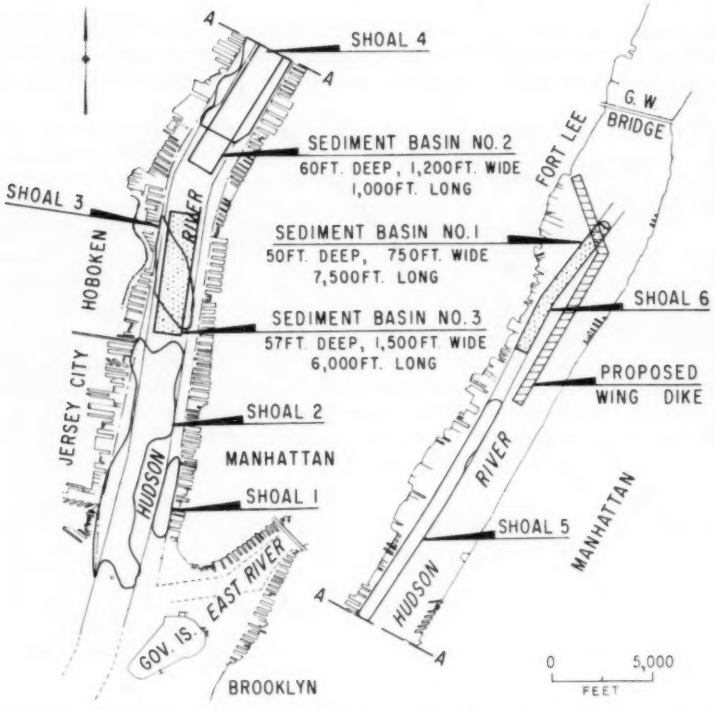


FIG. 10.—BASIN AND DIKE TESTS

capacity of about 4,000,000 yd each. Although they trapped enough material over a period of a prototype year to show reductions in the existing shoal areas of 10% to 70%, the total material deposited in the basins and elsewhere increased by about 50% over present conditions. This concept, by itself, is not considered to be a permanent solution, but has the advantages of a relatively cheap first cost and lower maintenance requirements.

As to channel realinement, the construction of the wing dike downstream from Fort Lee, N. J., to inclose the areas of heaviest shoaling did not prove effective. Although silt was effectively prevented from entering the problem

area from upstream, the flushing action of the tidal prism in the enclosed area was reduced to such an extent that a 50% increase in sedimentation occurred. For this and other reasons, this solution has been eliminated from further consideration.

An attempt to shift the natural deep-water channel from the New York side to the New Jersey side by filling the existing natural deep channel was similarly unsuccessful. Although a reduction of shoaling in the downstream shoal areas was effected, this was offset by an increase in the shoal at the head of the 30-ft channel. As in other unsuccessful plans, the total deposition was increased about 50% over present experience.

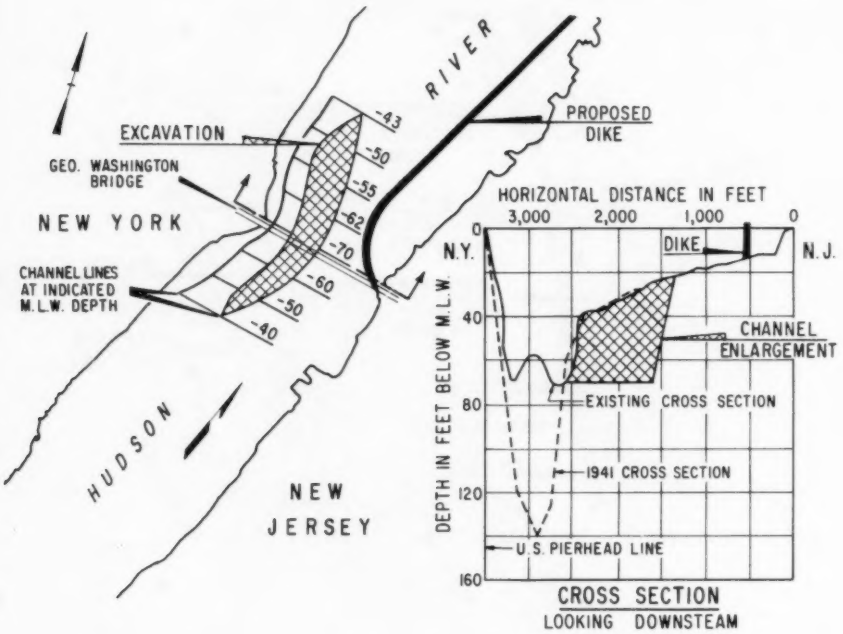


FIG. 11.—CHANNEL ENLARGEMENT TEST AT GEORGE WASHINGTON BRIDGE

Another scheme tested (Fig. 11) involved a change in channel geometry by increasing the cross-sectional area at the George Washington Bridge and by constricting the channel downstream by means of a dike. This resulted in a reduction of 50% in all shoal areas, except for the one at the upstream end of the 30-ft channel. Additional tests on variations of this concept are in progress in an effort to determine the maximum attainable benefits.

Another plan contemplated is the complete closure of the Harlem River by means of a movable gate at its junction with the Hudson, which would be closed on the flood tide to prevent flow toward the Hudson, but opened on the ebb tide to allow flow into the East River. This test indicated a reduction of 26% in

the project channel shoaling and demonstrated that bottom flow predominance in the vicinity of the George Washington Bridge is affected appreciably by the Harlem River's discharge. The adverse effect that this proposal would have on navigation on the Harlem all but precludes further consideration, however.

No meaningful tests have yet been run on the pier-slip model. However, the program includes the use of air and water jets across the slip entrances, as well as submerged and deflecting sills.

CONCLUSIONS

From the model studies that have been conducted to date at Vicksburg, it can be concluded that the extensive shoaling in the Hudson River, particularly in the Weehawken-Edgewater area, stems from three main sources:

1. The constriction in the cross-sectional area of the river at the George Washington Bridge.
2. An environment that is extremely favorable to the flocculation process.
3. The differential in depth between the artificial channel on the New Jersey shore and the natural deep-water channel on the New York shore.

Although no specific plan of improvement has been recommended to date, the most feasible corrective measures appear to relate to the construction of several large sedimentation basins, the enlargement of the river channel at the George Washington Bridge, and the provision of a wing dike from the New Jersey shoreline to constrict the channel south of the bridge. The recommended plan of improvement, in the final analysis, must be one that is economically acceptable, and as of this date, (1960), it would be premature to render judgment on the economic feasibility of any of the plans tested.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

MARINE OIL TERMINAL FOR RIO DE JANEIRO, BRAZIL

By H. W. Reeves,[1] M. ASCE

SYNOPSIS

A new oil loading and unloading terminal, under construction in Rio de Janeiro, Brazil (in 1960), is designed to accommodate tankers varying in size from 2,000 to 105,000 dead weight tonnage (DWT) capacity, and consists of a wharf structure in deep water connected by submarine pipelines to a refinery on shore. This paper describes the engineering design of the wharf, pipelines and mechanical equipment.

INTRODUCTION

There has been a remarkable increase in the size of ocean-going oil tankers. The T-2 tanker developed during World War II had a pronounced effect on the design of marine terminals throughout the world, and in the not-too-distant past it was felt that if a docking facility had been designed to accommodate a T-2 it was the ultimate in dock construction.

The T-2 tanker has a 16,500 DWT carrying capacity, a length of about 524 ft, and a full draft of about 30 ft. By 1949 the size of the largest tanker had increased to 31,000 DWT, and by 1954 to 45,000 DWT. In 1955 several tankers of 50,000 DWT class were under construction, and by 1957 two tankers of 84,750 DWT were in operation. The 1960's see the advent of the 105,000 DWT tanker. The approximate dimensions of the various tankers are shown in Table 1.

Note.—Discussion open until July 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 1, February, 1961.

[1] Staff Engr., Brown & Root, Inc., Houston, Texas.

Existing terminals and those to be built in the future must be patterned to accommodate these supertankers. No longer can the docking facilities for tankers be developed by using only age-old rules of thumb as the design basis. The design of a modern terminal to handle supertankers must be approached with the best scientific data and engineering know-how available. Every facet of the design must be investigated, and only after thorough research can the proper docking arrangement be developed. Such an arrangement should suit the particular physical harbor conditions and the characteristics of the tankers which will use the facilities. As the size of tankers increased, very few terminals in the world could handle the larger vessels without a modification of their existing facilities or the construction of new marine facilities in deep water. Since the modification of existing facilities by dredging to provide deeper water, is usually prohibitive in cost, new terminals with mooring facilities in deep water and served by underwater pipelines from the shore affords one of the better solutions to the problem. Some deep water mooring facilities include only a buoy-anchor mooring system, while others include a fixed island

TABLE 1.—DIMENSIONS OF TANICERS

DWT Tanker (1)	Length, in feet (2)	Beam, in feet (3)	Loaded Draft, in feet (4)	Approximate Capacity, in barrels (5)
16,500 (T-2)	523.5	68.0	30.2	145,000
26,500	628.5	82.5	32.0	225,000
30,155	659.5	85.0	34.25	252,000
38,000	707.0	93.0	36.5	318,000
45,000	736.5	102.0	37.25	393,000
50,000	782.0	102.0	37.5	425,000
84,750	850.0	125.0	46.0	707,000
94,000	950.0	128.0	48.0	800,000
105,000	950.0	135.0	50.0	1,021,000

type wharf structure for the berthing of tankers. The latter type of marine installation is under construction for Petrobras in Rio de Janeiro, Brazil (as of 1960), and is described in this paper.

THE GUANABARA MARINE TERMINAL, BRAZIL

The Brazilian oil company Petróleo Brasileiro S.A. (Petrobrás), in which the Brazilian government has controlling interest, is now constructing a marine terminal in the Guanabara Bay at Rio de Janeiro. The purpose of this terminal is to serve the 100,000 barrel per day Duque de Caxias Refinery, also under construction, by (1) unloading crude oil from tankers and transporting it by pipelines to the refinery; and (2) loading refined products from the refinery into tankers for both domestic distribution and foreign export.

The source of Brazil's oil production is located primarily in the state of Bahia some 1,000 miles north of Rio de Janeiro (Fig. 1). Crude oil produced

FIG. 1.—LOCATION MAP

in Bahia will be transported by tanker to the Duque de Caxias Refinery in Rio. Additional crude oil will be imported from foreign sources, primarily the Middle East.

The Guanabara Oil Terminal is comprised of facilities in three different areas (Fig. 2): the Duque de Caxias Refinery, Ilha d'Agua, and the tanker loading dock. Terminal facilities in the refinery area include fuel oil pumps, white product pumps, storage tanks, packaged steam generator units, communications, fire protection system, pig scraper traps, piping and valves.

The facilities on Ilha d'Agua include fuel oil pumps, white product pumps, a ballast treating system, storage tanks, electrical power generation systems, a terminal control center, an office and laboratory, a warehouse, an instru-

FIG. 2.—LOCATION DETAIL

ment and utility air system, packaged steam generator units, a water treating system, communications, positive displacement flow meters, a fire protection system, a barge loading facility, pig scraper traps, piping, and valves.

The facilities at the tanker loading dock include an island wharf structure which will berth two tankers, varying in size from 2,000 DWT to 105,000 DWT capacity simultaneously, two banks of hydraulic marine oil loading devices, each having six loading arms, drain sump and sump pumps, communications, pig scraper traps, a fire protection system, necessary lighting and navigational lights, piping and valves (Figs. 3 and 4).

All of the facilities for the Guanabara Terminal have been designed to have maximum operation flexibility for both crude and product movement with centralized data gathering facilities located in the terminal control center on

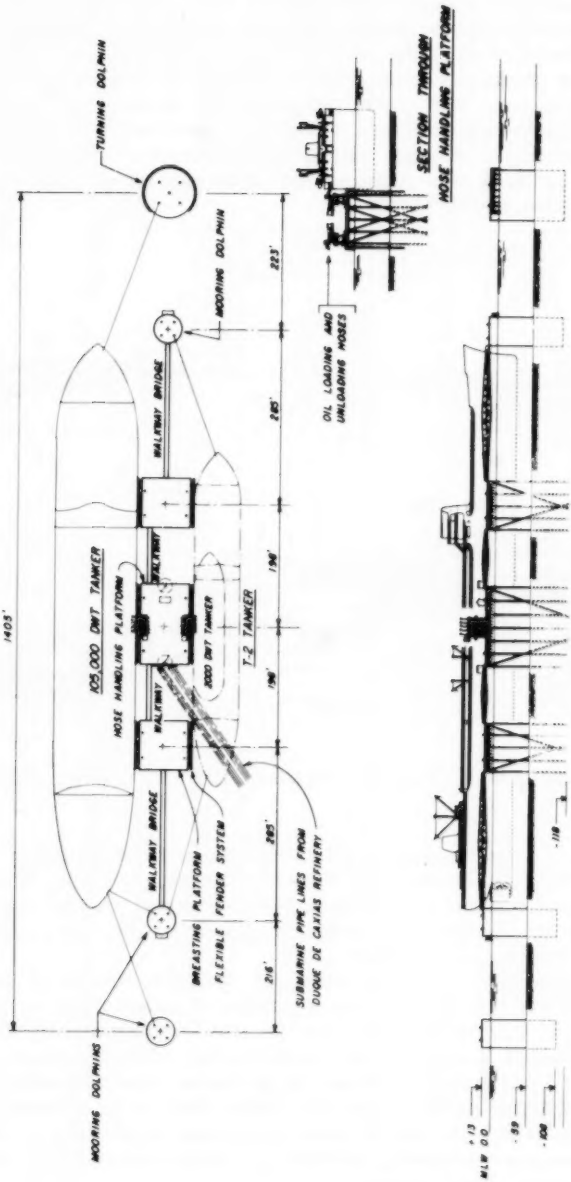


FIG. 3.—ISLAND TYPE WHARF AT RIO DE JANEIRO, BRAZIL

Ilha d'Agua. From the control center certain critical functions of the terminal can be controlled. By use of a flow-graphic panel at the control center, product movement can be followed. The black and white products pumps and the motor-operated valves can be controlled from this board. By being able to control the motor-operated valves from the panel watching the specific gravity recorder one can obtain minimum product contamination. The panel also includes remote counters with ticket punchers that obtain their signal from transmitters located at the positive displacement meters on Ilha d'Agua. The oil temperatures in the storage tanks, located at the refinery and on Ilha d'Agua, as well as the crude oils and fuel oil flowing temperatures are recorded on the control panel.

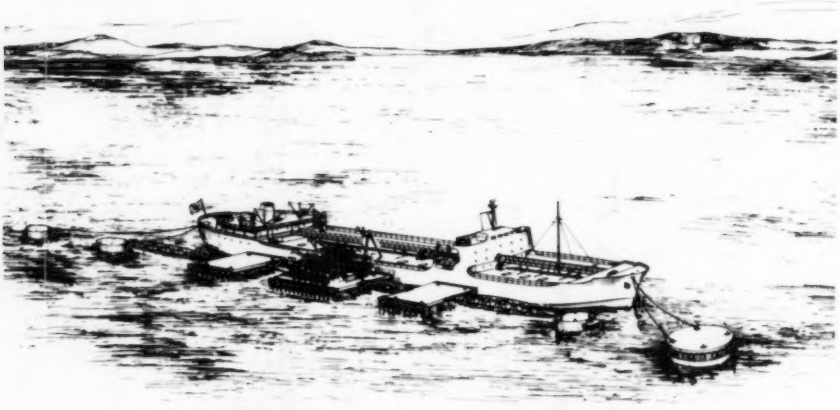


FIG. 4.—PETROBRÁS GUANABARA TERMINAL RIO DE JANEIRO, BRAZIL

The crude oil and product movement are shown on the schematic flow diagram (Fig. 5). This diagram does not show any valving, pig traps, instrumentation or the exact number of pipelines but is intended to show strictly the basic movement of the different products and crude oil. The systems are described in the following paragraphs.

Crude Oil Unloading.—Crude oil unloading facilities consist of two 26 in. submarine pipelines that extend from an offshore tanker dock, across Ilha d'Agua and Ilha do Governador and into the tank farm area of the Duque de Caxias Refinery, where a connection is made to the refinery storage tank fill headers. The two lines are tied together at the tanker dock with a bypass and block valve so that two ships delivering different crudes can be unloaded simultaneously. Motive power for the unloading operation is provided in all cases by the tanker pumping equipment, and no crude oil unloading pumps are provided under the scope of this project.

After the crude oil unloading is finished, it is necessary, due to the highly viscous Lagomar and Bahia crudes, to displace the pipe lines with flushing

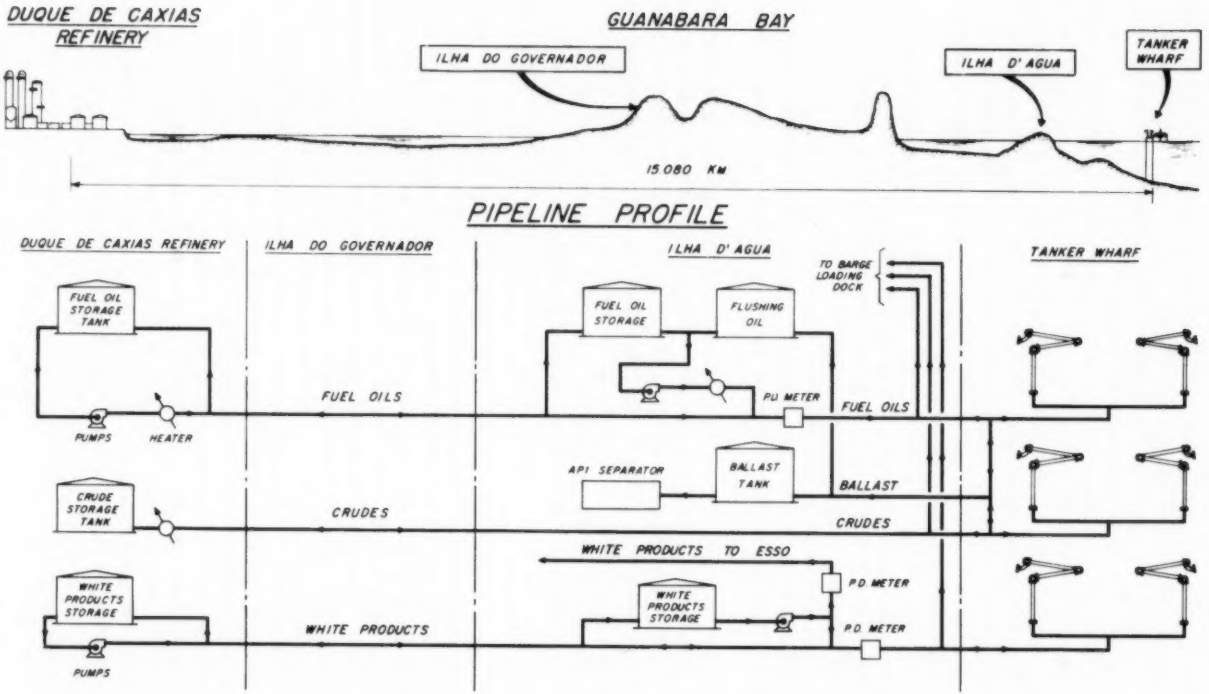


FIG. 5.—SCHEMATIC FLOW DIAGRAM

oil or recirculate the crude oil. The crude oil is kept at a temperature well above the pour point by circulating it through a set of heat exchangers located on Ilha d'Agua and at the refinery. The heating medium is steam from the steam generators. The submarine and land pipe lines have been given two coats of coal tar enamel and two spiral wraps of fiber glass wrap and in addition the submarine lines will be weight coated.

Fuel Oil.—The fuel oil system has been designed to transport fuel oil from the refinery direct to the tanker dock, move fuel oil from the refinery to Ilha d'Agua tank storage, and to move fuel oil from Ilha d'Agua tank storage to either the tanker dock or the barge loading facilities at Ilha d'Agua. In addition, the system is arranged so that flow can also be in the opposite direction, that is, fuel oil can be moved from either the tank dock back to Ilha d Agua, or from Ilha d'Agua back to the refinery.

Due to the high viscosity of either the Bahia base or Lagomar base fuel oils, it has been necessary to install heating facilities at the refinery's black products pump vicinity and at Ilha d'Agua. The above-ground lines have all been insulated with 85% magnesia insulation.

Pig traps are provided at the refinery, at Ilha d'Agua, and at the tanker dock so that cleaning may be accomplished, and connections are provided with motor operated valves so that the lines may be flushed to displace the fuel oil.

Flushing the line between the refinery and Ilha d'Agua may be accomplished from either direction utilizing the pipe-line pumps. Flushing from Ilha d'Agua to tanker dock is to be done by utilizing Ilha d'Agua flushing-oil storage and fuel-oil pumps and pumping out to the tanker dock and returning through the ballast line. Motor-operated valves are provided for rapid switching to minimize fuel-oil contamination. In addition, positive displacement meters are provided at both sides of Ilha d'Agua. They will automatically totalize flow in either direction on the flow graphic panel, located in the control room on Ilha d'Agua.

White Products.—The white products are pumped from the refinery direct to the terminal dock, to the oil company (Esso), or to the barge dock on Ilha d'Agua; from the refinery to Ilha d'Agua storage; from the tanker dock to either Ilha d'Agua storage, to the refinery, to the oil company, or to the barge dock; and from Ilha d'Agua to either the refinery, the tanker dock, the oil company, or the barge dock. The basic difference lies in the fact that a diversity of products is handled and that provision is made to move white products to the oil company's marine terminal on Ilha do Governador via a submarine pipeline.

For ease of operation, critical valves are provided with electrical operators, and position indication of the motor operated valves and other manually operated valves is registered on the flow graphic panel.

Product contamination has been held to a minimum by the installation of duplicate suction lines, rapid operating double valves, and warning devices. As is always the case where common pumps or lines are used to carry different products, some contamination will result, but Terminal da Guanabara has been designed to keep this to a minimum. By careful scheduling of the order in which products are put into the lines, blending will be done rather than down-grading or contamination.

For convenience, specific gravity meters are located in the lines for each product identification with remote recording and indication shown on the control room panel located on Ilha d'Agua.

Ballast Handling.—Complete facilities have been included at Ilha d'Agua for receiving and treating of ballast from tankers. These facilities include bypasses on the hydraulic marine crude oil unloading arms located at the tanker dock to receive ballast from ships, a submarine pipeline to handle ballast from the tanker dock to Ilha d'Agua, a ballast settling tank complete with skimmer pipe, air agitation distribution spider supplied from two stationary 600 cfm rotary air compressors, steam heating coils, and a two-compartment gravity differential separator with an effluent hay-basket filter.

Data Transmission.—Data transmission or telemetering of desired functions of the crude oil system consists of transmitting and indicating at the control room at Ilha d'Agua the positions of the valves admitting crude to the eighteen storage tanks and at the refinery, the valves admitting flushing-oil storage to the pumps and the tie-in block and bypass valves at the junction of the pipeline and refinery header system. In addition, the levels of the eighteen storage tanks are also transmitted to the control room at Ilha d'Agua.

The telemetering equipment for valve-position indication has been designed so that the system will scan all valves consecutively, one time, and indicate by means of memory lights on the flow graphic control panel on Ilha d'Agua the position of these valves.

The tank level indications have been designed so that from Ilha d'Agua any particular tank can be dialed and its level will be relayed back to Ilah d'Agua. The system is further arranged so that it can be put into an automatic scan condition and all tank levels will be monitored sequentially and their levels transmitted, or any one tank can be dialed or selected and the system will remain locked in order to monitor a filing or dumping operation occurring at this particular tank.

The data transmission system will function via an integrated telemetering selector bank which will receive input signals from valve limit switches and tank gaging telemetering transmitters and will transmit these inputs via a pulse code to the microwave system, where it will be microwaved to a receiver located at Ilha d'Agua and decoded and fed into the flow graphic panel. The microwave facilities are selected to have a five-frequency band so that two frequencies will be utilized for valve position indicating signal, one frequency for level indication, one channel for voice (not VHF), and one spare.

Fire Protection.—Fire protection is provided for Ilha d'Agua storage tanks and pump house by means of an automatic foam system, by dry chemical type fire extinguishers for the generator and control building and, at the warehouse and shop building, by an automatic wet pipe sprinkler system.

The entire tank farm area and the process and utility areas are provided with a salt water fire loop, pressurized by an automatically starting electric driven booster pump. In addition, the tank farm area and pump house are served by a foam loop pressurized by an automatic motor-driven pump. The tanker dock is protected by a manually started motor-driven fire pump pressurizing a fire loop.

Communications.—Communication throughout the Guanabara Terminal is maintained by means of very high frequency (VHF) radio equipment. These are located at the three different areas, the tanker wharf, Ilha d'Agua, Duque de

Caxias Refinery, and the Petrobrás head office in Rio de Janeiro. This design was made for maximum functional requirements with nominal expenditure for a project of this scope.

GENERAL DESCRIPTION OF WHARF STRUCTURE

The wharf structure is located, as shown on Fig. 2, about one kilometer from Ilha do Governador in approximately 60 ft of water, and the centerline of the wharf is oriented on a true north-south line. As shown on Fig. 3, the installation consists of a central hose and pipe manifold platform 132 ft long and 82 ft wide. The deck area of this platform is almost entirely taken up by pipe manifolds, valves, and the hose-handling equipment. Flanking to the north and south of the hose platform, and connected by 92-ft span walkway bridges to the hose platform, are 82-ft square breasting platforms. Mooring dolphins 46 ft in diameter are located as shown on Fig. 3. The mooring dolphins closest to the breasting platforms are connected to them by 222-ft span walkway bridges. At the southernmost terminus of the installation, and in the direction from which tankers will approach and depart, an 82-ft diameter turning dolphin is located.

The mooring dolphins and the turning dolphin are cellular sheet pile structures filled with sand and capped with massive concrete caps. The central loading and unloading platform, and the breasting platforms, are to have a reinforced concrete deck and are to be supported by either reinforced concrete or steel pipe piles.

One of the prime considerations in the design of this installation was the requirement that as much of the materials of construction as possible must either be produced in Brazil, or be readily importable from foreign countries with which Brazil has favorable trade conditions.

FIELD INVESTIGATIONS

In the investigations of physical factors which could influence the berthing of supertankers in Guanabara Bay, the following studies and investigations were made.

Wind Study.—The wind meterorological data were compiled from information furnished by the Ministério do Aeronáutica Station Galeão for the years 1951 to 1955, inclusive. These data showed a predominant wind from the south-south-east direction, varying in intensity from 0 to 33 knots.

Oceanographic Study.—This study included a detailed fathometer survey of the Guanabara Bay to determine the bottom characteristics of the approach channel, the recording of tide readings and ocean currents, and the determination of the characteristics of the maximum waves that could occur in the vicinity of the proposed wharf in Guanabara Bay. It was determined that the maximum tide is 1.6 m, but the average tide is somewhat less than 1 m. It was found that the ocean currents generally move in a north to south direction with the ebb tide, and reverse direction with the flood tide. The maximum current observed was 1.48 fps. Due to the fact that the Guanabara Bay is a well protected body of water, the maximum ocean waves that can develop in the bay area are less than 1.5 m in height and occur infrequently.

Soil Investigation.—With outcropping of granite rock showing prominently throughout the bay, and particularly in the area of the proposed wharf-structure location, one of the prime problems was to select a site on which a pile-supported structure could be constructed. To gather information about the foundation conditions at the proposed site, soil borings were made and continuous undisturbed soil samples were taken to the rock formation.

The first 20 ft to 25 ft of penetration disclosed an extremely soft, gray, highly organic clay. This soil is a sedimentary material, undoubtedly carried into Guanabara Bay by the Iguaçu and other small rivers which empty into the bay. The natural shearing strength (cohesion) of this material was found to be very low, and for design purposes no shearing resistance was assigned to this clay.

Underlaying this soft overburden is 25 ft to 30 ft of fine to coarse sand, in a medium to loose state, and having an angle of internal friction of 32° to 35° Underlaying the sand overburden is decomposed rock to an undertermined depth.

GENERAL DESIGN CONSIDERATIONS

A wharf structure to serve oil tankers should be designed to perform three functions.

1. Support the equipment necessary for the loading or unloading of crude oil or petroleum products.
2. Resist the mooring forces of tankers.
3. Resist the berthing or breasting forces of tankers.

To perform these functions, in contrast to a dry-cargo wharf, a modern marine oil-terminal wharf need not necessarily be designed as a single continuous pier or dock structure. Considerable economy can be realized by functionally designing separate structures to perform each of the previously listed three functions. A separate hose platform needs only to be the minimum size necessary to support all of the pipe manifolds, valves, hose handling, and other equipment required for the loading or unloading of crude oil or petroleum products. Since all of the pipe manifolds of supertankers are centrally located, only one hose platform is required per tanker berth.

Separate structures can be designed to resist the berthing and breasting forces from tankers. However, in some cases, designers prefer a single island structure which will perform as a hose platform and also as a berthing or breasting structure platform. To berth supertankers safely, the minimum out-to-out breasting length in contact with the ship is internationally accepted as one-third of the vessel's length; however, if this length can be increased, the lateral loads from breasted tankers and berthing forces from tankers under some berthing conditions can be materially reduced.

Berthing Forces.—In order to define the probable berthing forces of tankers properly, it is first necessary to evaluate some of the physical factors that will influence the probable berthing velocity of the vessels. These physical factors include ocean currents, winds, approach-channel characteristics, tides and the presence and magnitude of other traffic in the harbor. Indeed, it is not possible to set up one set of conditions or criteria for the determination of the probable berthing force of vessels that can be used for all pier designs. The type and

size of ship, how it is to be handled, and the harbor conditions as stated above, all influence berthing forces of ships. However, a designer should review some of the criteria used as design conditions for other wharf and dock designs as a guide in any new design.

Fig. 6 shows a limited number of berthing operation cases. Case I shows a vessel, apparently out of control, striking the wharf either laterally or in a frontal collision at an angle of impact greater than 15° with the face of the pier. This case represents a design condition that should not be considered in the design of the pier, since the cost of a pier to resist such forces would be prohibitive. The risk of damage from this type of collision must be covered by insurance.

Case II shows a vessel with a forward velocity and either under its own power or under the control of tugs approaching the pier and colliding at an oblique angle of 15° or less with the face of the pier. This case represents a design condition that should be accepted and considered in the design of the pier. This collision is mainly caused by some unforeseen circumstances during the berthing operation. Generally this case coincides with the maximum impact velocities that are subsequently examined.

Case III shows a vessel with a lateral velocity and under the control of tugs approaching the pier and colliding at an oblique angle less than 15° with the face of the pier, just before berthing broadside. This case represents a design condition that should be accepted and considered in the design of the pier. This collision could be caused as a result of circumstances similar to those mentioned in Case II.

Case IV shows a vessel completely under control of tugs approaching the pier in an ideal berthing operation and making contact with the pier entirely broadside.

The formula for determining the kinetic energy that is transmitted to the pier and which causes a berthing force to act transversely to the pier is.

$$E = E_O \rho \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which

$$E_O = \frac{T V^2}{2 g} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

T is the gross weight tonnage of the vessel, V represents the vessel velocity normal to face of pier, g denotes the acceleration of gravity, and ρ is a reduction coefficient. The determination of the value of this reduction coefficient is usually empirical. This reduction coefficient represents the percentage of E_O which causes a force to be transmitted laterally to the pier structure. There has been some investigation on this subject.

H. T. Horsfield[2] made a series of studies on this subject in 1948 using a small paddle steamer as the vessel and found that the kinetic energy absorbed by the pier represented from 41% to 72% of the total kinetic energy of the vessel.

R. R. Minikin[3] also made a series of studies on this subject from model experiments "wood on wood impact in quiet water and free," and found that from 18% to 27% of the vessel's kinetic energy was absorbed by the jetty.

---

[2] "Impact of a Vessel with a Pier," by H. T. Horsfield, Dock and Harbour Authority, London, June, 1948.

[3] "The Design of Jetties," by R. R. Minikin, Dock and Harbour Authority, October, 1943, June, 1947, and June, 1948.

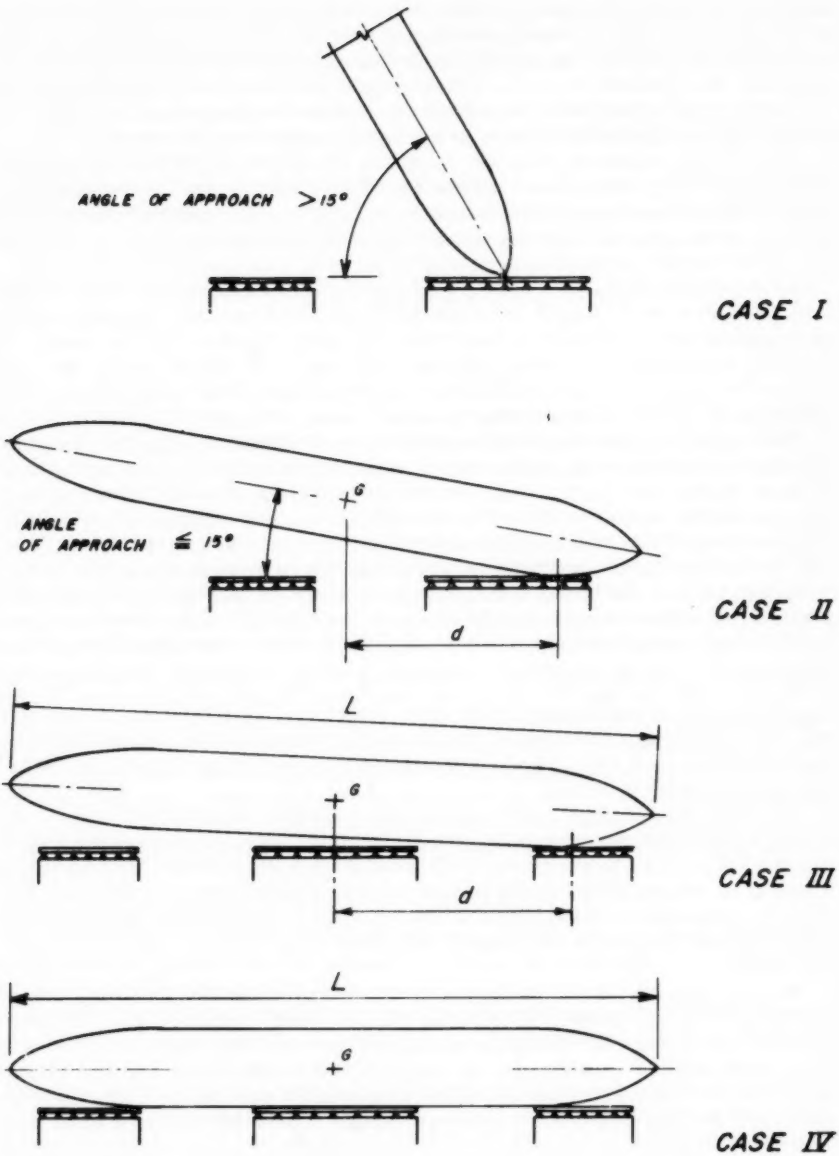

ANGLE OF APPROACH >15°
CASE I
ANGLE
OF APPROACH ≦ 15°
G
d
CASE II
L
G
d
CASE III
L
G
CASE IV

FIG. 6

Minikin further concluded that the mass of the water impelling the vessel forward under some circumstances can be a considerable influence on the blow delivered to a jetty. He recommends that a value of 50% be used as an energy reduction factor where steel vessels and reinforced concrete jetties are involved. D. A. Hopkins[4] suggests that a reduction coefficient value of 40% be used for pier designs. A. L. L. Baker[5] suggests that only 40% of the mass of a vessel is effective when movement of a vessel is stopped as it swings or rotates and collides with the pier at a point towards a bow or stern.

The French engineer, Pages[6], suggests the following formula for determining the energy reduction coefficient ρ. This formula uses the relationship between the tanker's approximate center of gravity, total length of vessel, and location of the point of berthing impact on the hull of the vessel.

$$\rho = \frac{1}{1 + 16\,\alpha^2} \qquad (3)$$

in which

$$\alpha = \frac{d}{L} \qquad (4)$$

and d and L are as shown on Fig. 6.

The value of d, which is the distance between the point of impact of the pier with the hull of the vessel and the vessel's approximate center of gravity, plays a most significant part in the determination of the amount of the vessel's energy that is transmitted to the pier structure. For example if the vessel strikes the pier at a point on the hull opposite the center of gravity of the vessel, the value of "d" would be zero and ρ would be 1.00. In other words, for this case 100% of the vessel's kinetic energy would be transmitted to the pier structure. On the other hand, if the extreme bow or stern of the vessel strikes the pier and assuming that the center of gravity of the vessel is exactly midship, $d = \frac{L}{2}$ and therefore $\rho = 0.20$. Thus for this case 20% of the vessel's kinetic energy is transmitted to the pier structure. Therefore, it can be seen that the percentage of vessels' kinetic energy which can be transmitted to a pier structure can vary between the values 20% and 100% of the total kinetic energy of a berthing vessel.

Checking the actual curvature of the bow and stern of numerous vessels up to 105,000 DWT displacement, the maximum values of α, for tankers striking the pier as in Case II, were computed and found to vary between 0.35 and 0.41. From this, the reduction coefficient was found to vary between 0.27 and 0.34. For the computation of the impact of tankers up to 105,000 DWT displacement colliding with the pier in accordance with Case II, an average value of $\rho = 0.3$ was used.

For the computation of the impact kinetic energy of the 105,000 DWT tanker colliding with the wharf, as in Case III, an energy reduction of coefficient $\rho = 0.56$ was computed. Using this same berthing condition, the energy reduction coefficient can be computed for the assumed condition that a single island structure having a total breasting surface equal to one-third of the length of the tanker is substituted for the present design. Under this berthing condition

---

[4] "The Design of Piers, Jetties and Dolphins," by D. A. Hopkins, Proceedings Separate No. 727, ASCE, June, 1955.

[5] Section II, Question 2, paper by A. L. L. Baker at XVIIIth Internatl. Navigation Congress, Rome, 1953.

[6] Paper by Pages in Annales des Ponts et Chaussées, March-April, 1952.

the energy reduction coefficient was computed to be $\rho = 0.78$. Therefore, it can be seen that the length of the wharf does affect the berthing energy transmitted to the wharf by a vessel under some berthing conditions. Under the berthing conditions of Case III the present design of the wharf as compared with a single-island structure resulted in a 22% reduction of the design berthing energy to be absorbed by the structure.

Under berthing conditions as shown in Case IV, the tanker will approach the pier and meet it entirely broadside. The energy-reduction coefficient in this case is $\rho = 1.00$.

A summary of the energy reduction coefficients of a 105,000 DWT tanker for the berthing cases shown on Fig. 6 shows that: for Case II $\rho = 0.30$; for Case III $\rho = 0.56$; and for Case IV $\rho = 1.00$.

The most critical factor in the determination of berthing impact loads of tankers is the design approach velocity of the vessel. The significance of this design approach velocity is tremendous, since the impact force increases by the square of that velocity. Improper selection of the design approach velocity can either result in a pier that will fail under berthing impact forces which should have been considered or, conversely, be many times stronger than need be to withstand the actual berthing forces satisfactorily. In view of the increased sizes of tankers, it would be most valuable to have some additional investigation or research concerning normal approach velocities and appropriate energy-reduction factors.

The Swedish engineer, Paul Leimdorfer,[7] made a series of observations of the berthing maneuvers of tankers and developed the velocity and energy curves (Fig. 7) for tankers up to 30,000 tons total displacement. The velocities of berthing V_a, V_b, and V_c coincide with various categories of wind speeds. The berthing velocity V_a applied to observations made with the wind speed less than 10 mph, and represented 12% of the total observations made. The berthing velocity V_b applied to observations made with the wind speed greater than 10 mph but less than 33 mph, and represented 70% of the total observations made. The berthing velocity V_c applied to observations made with the wind speed greater than 33 mph, and represented 18% of the total observations made. Leimdörfer further observed that the wind-speed frequency very often coincided with the respective berthing cases as shown on Fig. 6. Referring to the berthing cases as illustrated on Fig. 6 Case II coincides with impact speed V_c, Case III coincides with impact speed V_b and Case IV coincides with impact speed V_a.

The conclusions drawn from these observations are as follows:

1. In general, the larger the vessel, the smaller the impact speed. With the larger tankers, greater care is usually taken in the berthing operation than with smaller tankers.

2. The larger supertankers are less dependent on weather conditions and consequently the curves V_a, V_b, and V_c tend to get minimum values of about 10 fpm, 15 fpm, and 25 fpm respectively.

3. The curves E_a, E_b, and E_c, representing the kinetic energy developed by tankers up to 30,000 tons total displacement, show that the maximum E_a, E_b, and E_c values were developed by 23,500, 22,500, and 16,500 ton displacement tankers, respectively. The conclusion drawn from these observations

[7] Section II, Question 2, paper by Paul Leimdörfer at XIXth Internatl. Navigation Congress, London, 1957.

was that, because of the usually greater design approach velocities for smaller vessels, the berthing impact forces created by larger vessels may not be appreciably larger than the forces created by smaller vessels.

The velocity and kinetic energy curves as presented by Leimdörfer include tankers only up to 30,000 tons total displacement. Since tankers of greater size

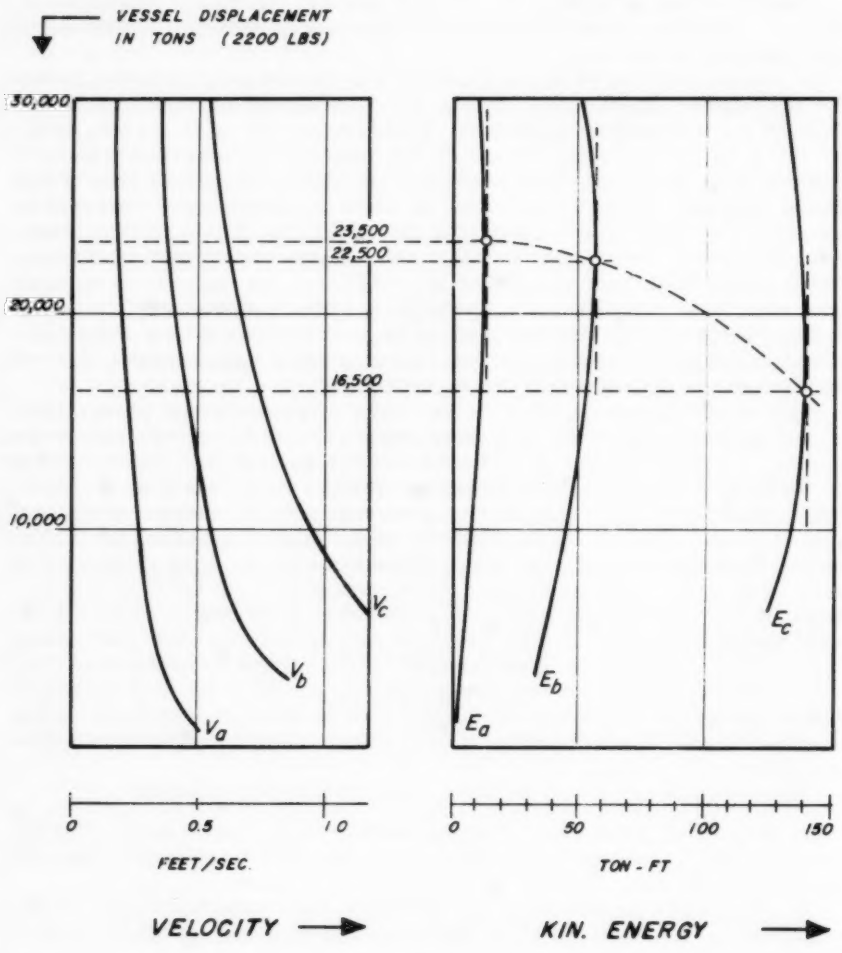


FIG. 7.—VELOCITY AND ENERGY CURVES

will be using the Guanabara Terminal, the velocity curves as presented by Leimdörfer were extended to include these larger vessels. Using these extended velocity curves the berthing kinetic energy curves for tankers were plotted as shown on Fig. 8.

Referring to Fig. 8 it can be seenthat the 105,000 DWT tanker will develop a berthing kinetic energy of 62 ton-ft for berthing Case IV, 135 ton-ft for berthing Case III, and 370 ton-ft for berthing Case II.

To determine the kinetic energy transmitted to the wharf structure for the various berthing cases we can apply the previously derived energy reduction coefficients which yields the following values.

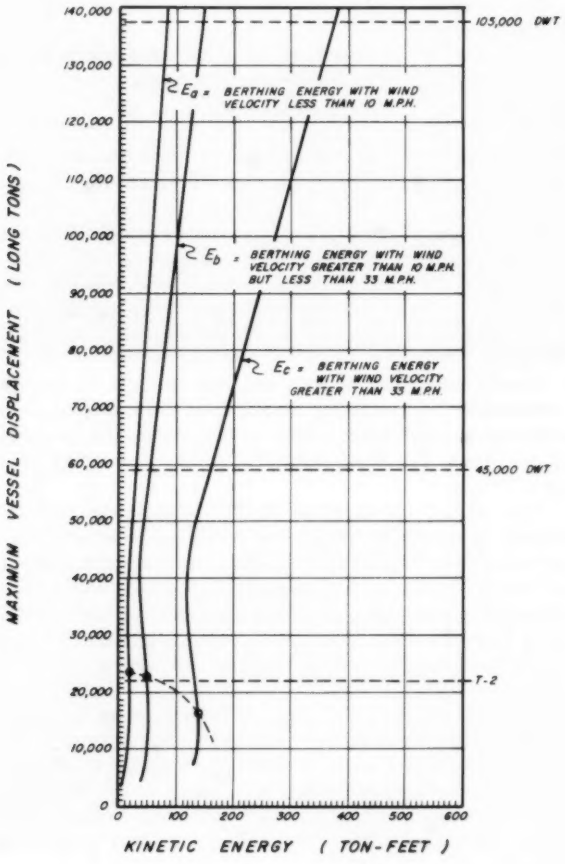


FIG. 8.—BERTHING KINETIC ENERGY CURVES FOR TANKERS (AFTER LEIMDÖRFER)

For Case II ρ = 0.30; berthing energy to wharf = 111 ton-ft
For Case III ρ = 0.56; berthing energy to wharf = 76 ton-ft
For Case IV ρ = 1.00; berthing energy to wharf = 62 ton-ft

Breasting Forces.—Even after a vessel is safely moored and tied up at a wharf, it can still subject the wharf structure to a breasting force of con-

siderable magnitude. This force is created by winds blowing on the projected area of the vessel above the water surface, and by waves and ocean currents acting on that portion of the vessel below the water surface.

The French engineer, P. Callet, has made considerable study of these forces and some excellent information can be derived from various papers he presented before the Permanent International Association of Navigation Congresses. However, in the case of this location in Guanabara Bay, these breasting forces are considerably smaller than the berthing forces previously discussed. For that reason, it was not necessary to go into great detail in the evaluation of these forces.

The wind force on the projected area of the ship above the water surface was determined by the U.S. Weather Bureau formula

$$F_W = 0.004\ V^2A \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

in which V is the wind velocity, in miles per hour; A denotes the exposed projected area of vessel, in square feet; and F_W refers to the total breasting force from wind, in pounds.

Due to the fact that the tidal currents and waves are small in Guanabara Bay, and since they would act along the longitudinal axis of the moored vessel, they were ignored in the determination of the total breasting force.

Mooring Forces.—There have been considerable numbers of papers presented on the subject of mooring forces, the most recent of which (as of 1960) is by B. W. Wilson.[8]

For this wharf, however, it was decided merely to design the mooring bollards to withstand the breaking strength of the mooring lines. This force was arbitrarily set as 100 tons, which is more than the breaking strength of the mooring lines; thus, the lines would fail before damaging the mooring bollard and dolphin.

Fenders.—The design of flexible fender systems to resist the berthing forces of vessels is one of the most neglected considerations in the planning of marine installations. Fendering of port installations was for many years considered a necessary evil, the main purpose of which was to protect the pier from being damaged by berthing vessels, and which actually did little to neutralize the heavy forces acting upon the piers and vessels. More often than not a pier would be completely designed before a fendering system was decided upon.

This attitude in design is on its way out. A better understanding of the forces involved, and better methods of handling them, have given engineers and designers good reason to start with the fendering system when designing a pier. By doing this, they are able to take full advantage of the efficiency of the system, and tailor the pier structure to meet the requirements of forces greatly reduced by this absorption. The result is a better and frequently less expensive installation.

Designers have also realized the tremendous advantage of fendering when existing piers have to be revamped to accommodate larger vessels than the original design called for. Frequently the pier strength is sufficient as it stands, and little or no structural changes are necessary, provided it is given the added protection of an efficient fendering system. The fendering absorbs the extra energy of the larger vessels, and the pier loading figure remains the same as it was originally, when the smaller vessels berthed without fendering.

---

[8] "A Case of Critical Surging of a Moored Ship," by Basil Wilson, Proceedings, ASCE, Vol. 85, No. WW 4, December, 1959.

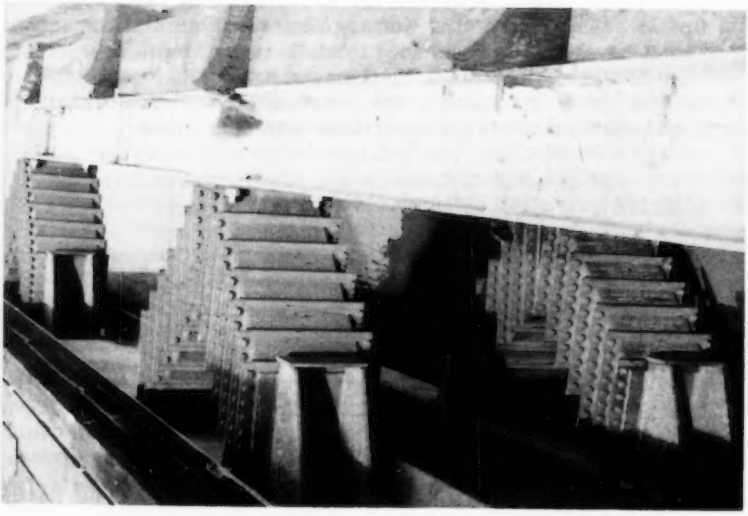


FIG. 10.—RUBBER SANDWICH BUFFER INSTALLATION

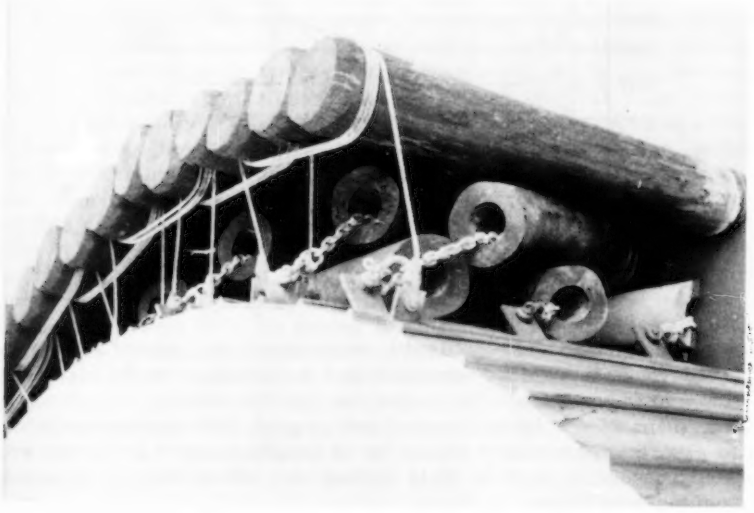


FIG. 9.—RUBBER TUBE FENDER INSTALLATION

In the past, with smaller tankers to be berthed than are now in service, wood fender piles and camels performed satisfactorily. This system employed the use of timber piles driven along the face of the wharf with floating camels placed in the water in front of the timber piles to make initial contact with the tanker as it berthed. The resistance to the berthing forces was provided by the energy dissipated in pushing the camels through the water and compressive deformation of the timber. The low energy-absorption capacity for this system makes it unsuitable for supertankers in most cases.

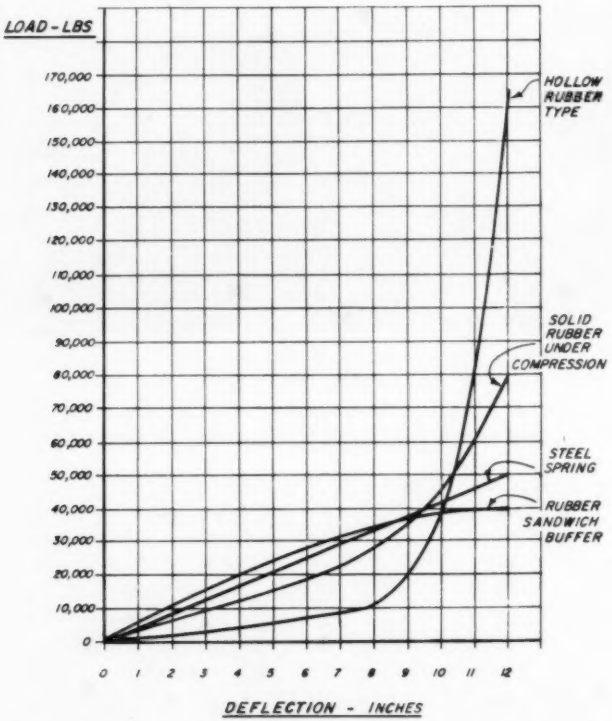


FIG. 11.—LOAD DEFLECTION CURVES FOR VARIOUS FENDERING SYSTEMS WITH EQUAL ENERGY ABSORPTION AT 12 INCH DEFLECTION

Another system that has been used to some extent are gravity and inertia fenders. This system employs a heavy fender suspended from the structure. As the ship makes contact with the fender, the berthing energy is absorbed by moving the mass of the fender inward and upward. This system can be designed to absorb any amount of energy but is usually massive and requires a complicated suspension system. This system also offers little resistance to longitudinal berthing forces.

Another system that is used quite widely incorporates the use of various types of springs. Some of the springs in use are steel leaf and coil springs, rubber blocks, rubber tubes (Fig. 9), and rubber-sandwich buffers (Fig. 10).

Fig. 11 shows typical load-deflection curves plotted for steel springs, rubber tubes, solid rubber, and rubber-sandwich buffers under compression loads with equal absorption at 12 in. deflection.

Steel springs can be designed to absorb practically any amount of energy, but structural guides must be provided to protect the springs from longitudinal berthing forces, and some limiting devices must be employed to prevent the fender-wale system from pulling away from the springs. From all reports, existing steel-spring systems have performed satisfactorily.

Rubber blocks have been used on some installations. The details of many such installations are described in detail by D. H. Little.[9] This design uses a conventional timber pile and wale system that utilizes the rubber blocks to provide the buffer action between the pier and pile-wale system. The rectangular solid rubber blocks are sandwiched between two steel plates. The plates are bonded to the rubber and are bolted to the pier and the timber fender system. This system generally does not provide sufficient energy absorption capacity, and reference to Fig. 11 shows that the load-deflection curve is unfavorable to proper fendering.

Rubber tubes have been used extensively in fender systems. The rubber tubes are shaped like a hose but with a thickened wall. The most conventional way of using this system is to string the rubber tubes horizontally or vertically on the face of the pier behind a pile-wale system. Rubber tubes used in this manner do not have a linear load-deflection curve. The rubber tube absorbs very little energy until the tube closes, then it absorvs energy at a greatly increased rate. Rubber tubes, however, by their shape, lend themselves very well to the fendering of turning dolphins and other circular sheet pile structures. Fig. 9 shows rubber tube fenders used in this manner. It is in this service that the rubber tubes are used on this project. The turning dolphin is fendered and protected against the impact loads from supertankers in the turning maneuvers by five rows of 18 in.-by-9 in. rubber tubes draped horizontally around the turning dolphin. Timber rubbing mattresses are suspended from the top of the turning dolphin to make initial contact with the ship and to transmit the load to the rubber tubes in contact with the turning dolphin.

It has been found by observation of other marine installations that, without the timber mattresses, ships will exert such tremendous sliding friction forces on the rubber tubes that they will, in some cases, be torn or the suspending chains or cables ripped off the turning dolphin. Timber has a lubricating effect on the sliding forces from ships and can be economically replaced as required. Rubber tubes have been used on some installations with end loading of the rubber tubes instead of the side loading condition as previously described. In this type of service the rubber tubes are cut to lengths of 1-1/2 times the outside diameter of the tubes and placed between the face of the pier and a pile-wale system. The load-deflection curve of rubber tubes used in this manner is practically linear. However, as in the case of steel-spring fenders, end loaded rubber tubes require limiting devices to prevent the fender-wale from pulling away from the rubber tubes and this system does not offer much resistance to longitudinal berthing forces. Fender systems involving end loaded rubber tubes, however, have been in use for several years and according to all reports have performed satisfactorily.

The fender selected for the breasting platforms of the Guanabara Terminal is a rubber-sandwich buffer. Fig. 10 shows rubber-sandwich buffers installed on a wharf structure which is similar to the Guanabara Terminal Wharf. This buffer consists of rectangular rubber sandwiches bonded to steel plates, which

[9] "Some Designs for Flexible Fenders," by D. H. Little, Proceedings, Inst. of C. E., London, February, 1953.

are bolted together to form an inverted V. This buffer does not require tension limit devices or guides, since it will work in tension as well as compression and will offer resistance to longitudinal loads. Another important characteristic of this buffer is that, due to the shape of its load-deflection curve, it will yield a lower reaction than a comparable steel spring at a given energy absorption and deflection.

It should be pointed out, however, that the starting point in any design is to determine the proper cost relationship between the pier and the fenders. Fender systems depend on the pier or wharf design. For instance, a flexible pier that will deflect under berthing forces will dissipate much of the vessel's kinetic energy. On the other hand, if a pier is rigid, little of the vessel's kinetic energy can be dissipated by the pier, and the fender system must be designed to absorb the berthing impact force. If the fender system is insufficient to perform the job, serious damage to the pier and to the ship can result. Further, a properly designed fender system may permit a less costly pier design if the fender is permitted to dissipate the load and properly distribute the reactions into the pier structure.

In designing various types of fender systems, the designer should not overlook the fact that sometimes the least expensive and most elementary designs are best for some installations. For example, the most effective fender developed to act as a buffer against barges and work boats mooring against offshore drilling platforms in the Gulf of Mexico is discarded automobile tires.

Fender Design Computations.—After considering all of the previously described physical factors for the proposed wharf location in Guanabara Bay and by working closely with the Marine Department of Petrobrás, the following design criteria for the berthing forces of tankers are developed. These berthing force calculations are based on methods previously described and the assumptions used in the computations are as follows:

For berthing Case II, as shown on Fig. 6, the approach velocity of the 105,000 DWT tanker normal to the face of the wharf was assumed to be 25 fpm. Under this berthing condition the vessel has an assumed forward velocity of one knot and the angle of approach of the vessel with the face of the wharf is 15°.

Size of tanker 137,140 tons total displacement
Approach Velocity 1.0 knots
Approach Angle 15°

$$\text{Lateral Approach Velocity} = \frac{(1.0 \text{ knot})\,(6080.2 \text{ ft per nautical mile})(\sin 15°)}{(3600 \text{ sec per hr})}$$

$$= 0.42 \text{ fps}$$

$$\text{Kinetic Energy (from Eq. 2)} \frac{T\,V^2}{2\,g} = \frac{(137{,}140 \text{ tons})\,(0.42 \text{ fps})^2}{(2)\,(32.2 \text{ fps})}$$

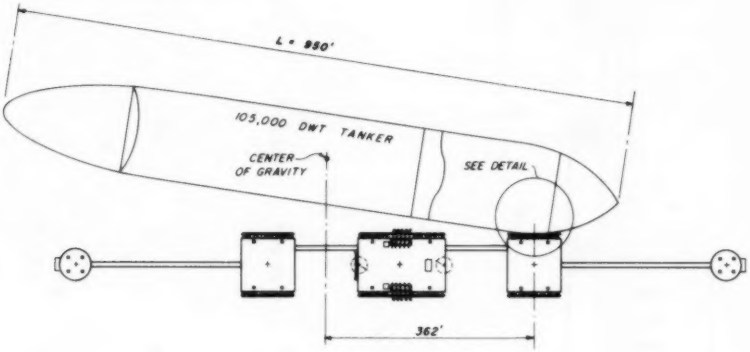
$$= 370 \text{ ft-tons}$$

$$\text{Energy (from Eq. 1 and Fig. 12)} = (370 \text{ ton-ft})\,(0.30) = 111 \text{ ton-ft}$$

The fender design calculations for this berthing condition are shown on Figs. 12 and 13. The actual curve of the bow of the 105,000 DWT tanker was compared with the load-deflection curve for the fender system under the design loading and for this condition it was determined that approximately 30 ft of

the bow of the vessel will be in actual contact with the fender system at the time of maximum deflection.

Referring to Fig. 12, it was calculated that the vessel will transmit 111 ton-ft in kinetic energy to the wharf structure for this berthing condition. The

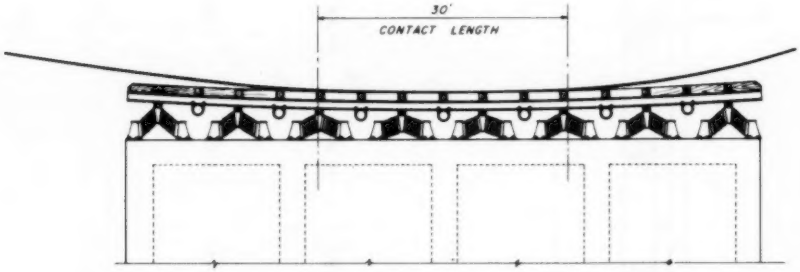


BERTHING CONDITION CASE II

REDUCTION COEFFICIENT CALCULATIONS

$$\rho = \frac{1}{1 + 16\alpha^4} \qquad \text{WHERE} \quad \alpha = \frac{d}{L} = \frac{362}{950} = 0.38$$

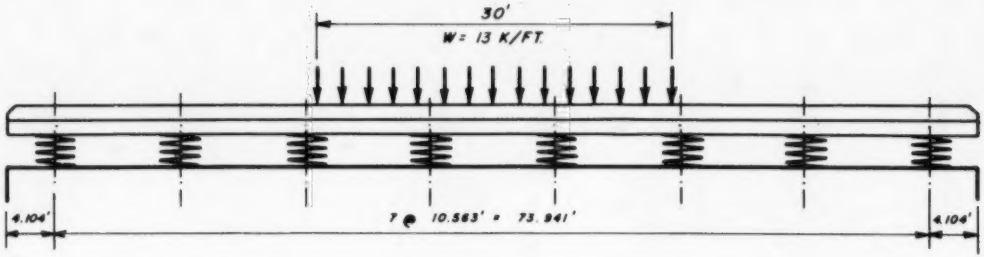
$$\rho = \underline{0.30}$$



DETAIL

FIG. 12.—FENDER DESIGN-CASE II

problem of equating the total energy absorbed by the fender system was solved by using solutions of beams on elastic foundations. In using this method for calculating the energy absorbed by the fender system, it was first necessary to establish the stiffness factor for each spring under load. Since the rubber

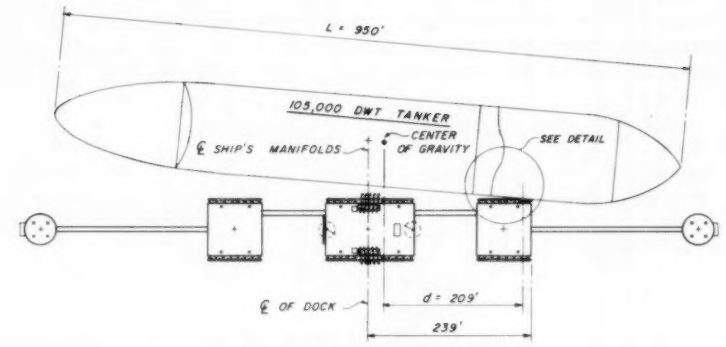


LOAD DIAGRAM

REACTION :	2.78 K	38.82 K	64.54 K	84.86 K	84.86 K	64.54 K	38.82 K	2.78 K
DEFLECTION :	0.043'	0.611'	1.142'	1.542'	1.542'	1.142'	0.611'	0.043'
ENERGY :	0.01 K'	12.1 K'	40.7 K'	71.6 K'	71.6 K'	40.7 K'	12.1 K'	0.01 K'

FIG. 13.—FENDER DESIGN

sandwich buffers selected for this project do not have a linear load-deflection curve, the stiffness factor for each spring varied according to the load. In order to establish the stiffness factor for the individual springs, it was first necessary to estimate the deflection curve for the entire fender system under

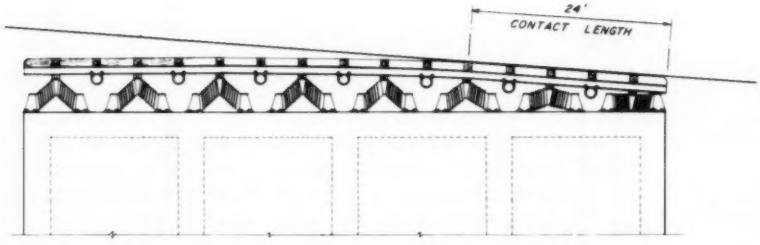


BERTHING CONDITION CASE III

REDUCTION COEFFICIENT CALCULATIONS

$$\rho = \frac{1}{1 + 16\alpha^2} \qquad \text{WHERE} \quad \alpha = \frac{d}{L} = \frac{209}{950} = 0.220$$

$$\rho = 0.56$$

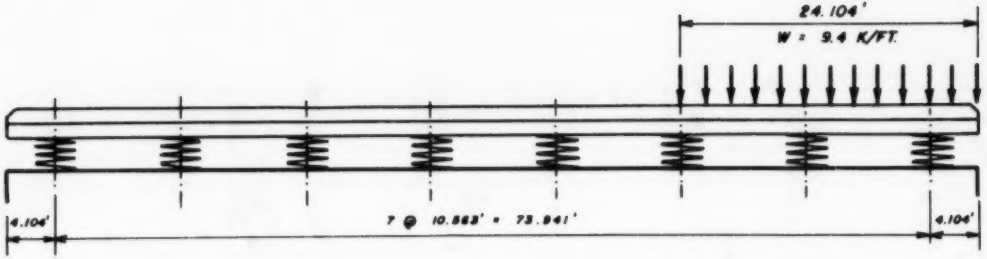


DETAIL

FIG. 14.—FENDER DESIGN-CASE III

load. Using the fender stiffness factors thus established, the total energy absorbed by the fender system was equated to the berthing energy transmitted by the vessel to the wharf structure.

As shown on Fig. 13 an assumed uniform loading of 13 kips per ft distributed over 30 ft of the fender face approximates the loading exerted by the vessel for this berthing condition.



LOAD DIAGRAM

REACTION :	-7.90 K	-5.46 K	-0.69 K	8.90 K	26.25 K	48.04 K	72.16 K	95.84 K
DEFLECTION :	-0.125'	-0.085'	-0.010'	0.14'	0.41'	0.765'	1.21'	1.87
ENERGY :	0.55 K'	0.25 K'	0.00 K'	0.70 K'	5.50 K'	18.75 K'	45.50 K'	101.00 K'

FIG. 15.—FENDER DESIGN

For berthing Case III as shown on Fig. 6 it was assumed that the vessel would be under the control of tugs and would make contact with the wharf structure while under a lateral velocity of 15 fpm. The fender design calculations for this berthing condition are shown on Figs. 14 and 15. In this berthing case the hose manifolds of the vessel would be more or less in line with the loading hose devices on the wharf structure and the flat side of the vessel would make contact with the flexible fender system as shown on Fig. 14.

Referring to Fig. 14, it was calculated that the vessel will transmit 76 ton-ft in kinetic energy to the wharf structure for this berthing condition.

$$E_O = 135 \text{ ton-ft (from Fig. 8)}$$
$$E \text{ (from Eq. 1)} = (135)(0.56) = 76 \text{ ton-ft}$$

As shown on Fig. 15 an assumed uniform loading of 9.4 kips per ft distributed over 24 ft of the fender face approximates the loading exerted by the vessel for this berthing condition.

CONCLUSIONS

In the design of a marine terminal many factors must be investigated and resolved and it is the purpose of this paper to outline some of these problems. The Guanabara Terminal in Rio de Janeiro, Brazil, is merely used as an example to show some of the various problems that can confront a designer. This paper covers only the engineering design phase for this terminal and the information and design information as presented represents the design of the terminal.

It is hoped that this paper will lead to discussion and more investigation of the berthing velocities of vessels and other factors so necessary for the proper design of modern marine terminals.

ACKNOWLEDGMENTS

Construction specifications and completely detailed drawings were prepared by Brown and Root, Inc., under the direct supervision of the author. The engineering for the entire project was under the direction of Paulo M. de Oliveria Castro, Manager of the Pipeline Department of Petrobrás.

The author wishes to thank Petrobrás and particularly Mr. Castro for permission to present this paper. Fig. 7 has been reproduced with the permission of the Permanent International Association of Navigation Congress.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

BEHAVIOR OF BEACH FILLS IN NEW ENGLAND

By Harry S, Perdikis,[1] M. ASCE

SYNOPSIS

A study has been made of the behavior of fills placed on ten New England beaches. The study involved quantitative determinations of changes in volumes, high and low water shore line positions, and slopes. Descriptions are given of construction of the projects, wave exposure, past erosion, littoral drift, and character of beach and fill materials. Comparison is made between estimated and experienced rates of change. The difficulty in predicting the behavior of beach fills from the behavior of the original beaches is pointed out.

INTRODUCTION

The placement of fill for the widening of beaches has in recent years, become a common method used by the engineer for the restoration and improvement of shore areas for recreational use and for their protection against wave attack. This method has been widely used along the New England coast.

Beaches in New England known to have been improved by placement of fill are listed in Table 1. The locations of the beaches are shown on Fig. 1, numbered as listed in Table 1.

Despite the widespread use of beach fills in New England, there has been little quantitative data available concerning their behavior. It has been apparent from visual observations that some fills remained comparatively unchanged for many years while others have experienced large changes in a period of a

Note.—Discussion open until July 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 1, February, 1961.

[1] Civ. Engr., Chf., Beach Erosion Studies, U.S. Army Engr. Div., New England, Corps of Engrs., Waltham, Mass.

TABLE 1.—NEW ENGLAND BEACHES IMPROVED BY FILL PLACEMENT

1. Hampton Beach, Hampton, N. H.
2. Plum Island, Newburyport and Newbury, Mass.
3. Wingaersheek Beach, Gloucester, Mass.
4. Singing Beach, Manchester, Mass.
5. Dane Street Beach, Beverly, Mass.
6. Salem Willows, Salem, Mass.
7. Collins Cove, Salem, Mass.
8. Palmers Cove, Salem, Mass.
9. Pioneer Village, Salem, Mass.
10. Forest Beach Park, Salem, Mass.
11. Front Street Beach, Marblehead, Mass.
12. Fishermans Beach, Swampscott, Mass.
13. Revere Beach, Revere, Mass.
14. Orient Heights Beach, East Boston, Mass.
15. Quincy Shore, Quincy, Mass.
16. Germantown Beach, Quincy, Mass.
17. Wessagussett Beach, Weymouth, Mass.
18. Between First and Second Cliff, Scituate, Mass.
19. Pegotty Beach, Scituate, Mass.
20. Brant Rock, Marshfield, Mass.
21. Chase Garden Beach, Yarmouth Mass.
22. Red River Beach, Harwich, Mass.
23. East of Herring River, Harwich, Mass.
24. West Dennis Beach, Dennis, Mass.
25. East and West of Parkers River, Dennis, Mass.
26. South Yarmouth Beach, Yarmouth, Mass.
27. Englewood Beach, Yarmouth, Mass.
28. Kalmus Park Beach, Barnstable, Mass.
29. Veterans' Memorial Park Beach, Barnstable, Mass.
30. Dead Neck, Barnstable, Mass.
31. Loop Beach, Cotuit, Mass.
32. Falmouth Heights, Falmouth, Mass.
33. Meganset Beach, Falmouth, Mass.
34. Wild Harbor, Falmouth, Mass.
35. Pocasset Beach, Bourne, Mass.
36. Monument Beach, Bourne, Mass.
37. Buttermilk Bay, Wareham, Mass.
38. Onset Bay, Wareham, Mass.
39. Little Harbor, Wareham, Mass.
40. Long Beach, Wareham, Mass.
41. Parkwood Beach, Wareham, Mass.
42. Pinehurst Beach, Wareham, Mass.
43. Hamilton Beach, Wareham, Mass.
44. Swift Beach, Wareham, Mass
45. Silver Shell Beach, Marion, Mass.
46. Water Street Beach, Mattapoisett, Mass.
47. Pope Beach, Fairhaven, Mass.
48. East and West Rodney French Blvd. Beaches, New Bedford, Mass.
49. Horseneck Beach, Westport, Mass.
50. Childrens Beach, Nantucket, Mass.
51. Sand Hill Cove, Narragansett, R. I.
52. Misquamict Beach, Westerly, R. I.
53. Napatree Beach, Westerly, R. I.
54. Eastern Point Beach, Groton, Conn.
55. Ocean Beach, New London, Conn.
56. Hawks Nest Beach, Old Lyme, Conn.
57. White Sand Beach, Old Lyme, Conn.
58. Chalker Beach, Old Saybrook, Conn.
59. Hammonasset Beach, Madison, Conn.
60. Jacobs Beach, Guilford, Conn.
61. West Silver Sands Beach, East Haven, Conn.
62. Lighthouse Point Park, New Haven, Conn.
63. Savin Rock, West Haven, Conn.
64. Prospect Beach, West Haven, Conn.
65. Woodmont Shore, Milford, Conn.
66. Gulf Beach, Milford, Conn.
67. Silver to Myrtle Beaches, Milford, Conn.
68. Laurel and Cedar Beaches, Milford, Conn.
69. Short Beach, Stratford, Conn.
70. Pleasure Beach, Bridgeport, Conn.
71. Seaside Park, Bridgeport, Conn.
72. Fairfield Beach, Fairfield, Conn.
73. Sasco Hill Beach, Fairfield, Conn.
74. Southport Beach, Fairfield, Conn.
75. Burial Hill Beach, Westport, Conn.
76. Sherwood Island State Park, Westport, Conn.
77. Compo Beach, Westport, Conn.
78. Calf Pasture Beach Park, Norwalk, Conn.
79. Cove Island, Stanford, Conn.

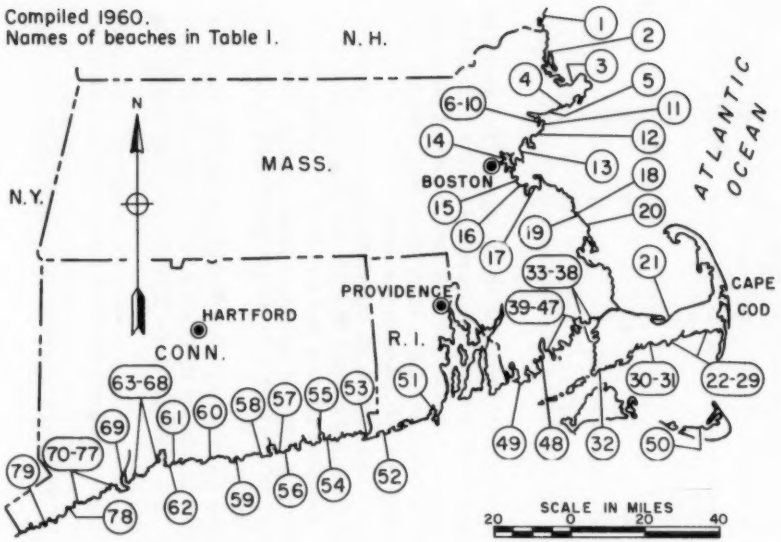


FIG. 1.—NEW ENGLAND BEACHES IMPROVED BY FILL PLACEMENT

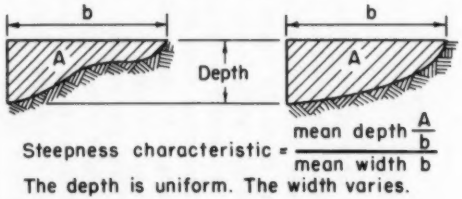


FIG. 2.—STEEPNESS CHARACTERISTIC

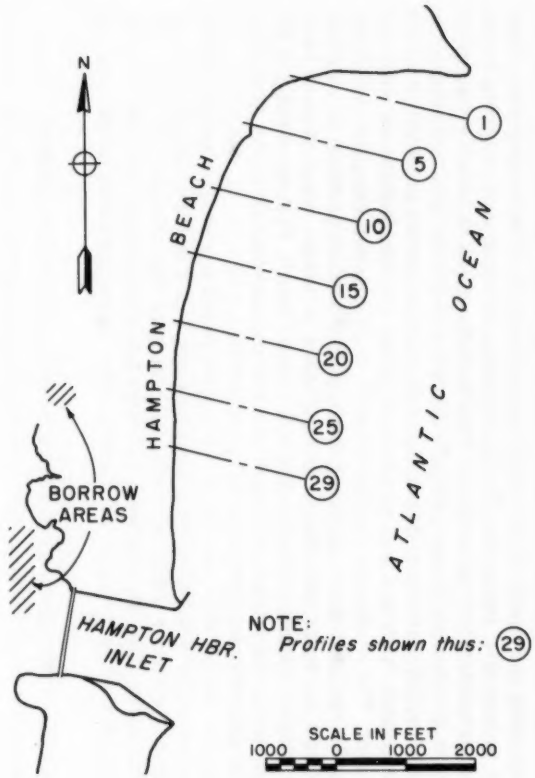


FIG. 3.—HAMPTON BEACH, N. H.

few months. Quantitative information under all types of conditions is needed for improving the design of beach fill projects and for determining when their use is practicable and economically justified. Information concerning beach fills is presented for ten projects. This includes dates of construction, wave exposure, erosion problems, probable direction of littoral drift, and character of beach and fill materials. Beach slope comparisons were made for the zone between mean high and low water at selected profiles before and after fill placement. The steepness characteristic[2] of Per Brunn, computed as shown on Fig. 2, was used due to the irregularities in the beach profiles which made it difficult to make meaningful comparisons of actual beach slopes. As in actual slopes, higher values indicate a steeper beach. Volume of fill and fill changes were computed from beach surveys and movements of the shore line were measured from comparative beach profiles.

Hampton Beach, Hampton, N. H.—Hampton Beach was widened by placement of approximately 400,000 cu yd of sand fill along 1-1/4 miles of its shore from August to December 1955, by hydraulic dredging from Hampton Harbor and Hampton River (Fig. 3). The work restored beach area which had been eroded over a period of years and it provided a wider beach for recreational use and

TABLE 2.—COMPARISON OF STEEPNESS CHARACTERISTICS HAMPTON BEACH

Profile	Steepness Characteristics		
	Before Fill June 1955	After Fill December 1955	After Fill January 1959
2	.020	.027	.015
9	.017	.020	.020
15	.014	.017	.023
21	.027	.026	.017
27	.012	.012	.013
Average	.018	.020	.018

for protection of the shore development from storm damages.[3] The shore is directly exposed to wave attack from the Atlantic Ocean. Littoral drift which moves predominantly from north to south had resulted in loss of all of the sand from the north end of the beach leaving a layer of gravel, cobbles, and boulders while lesser gradual losses continued to the south.

The sandy portion of the original beach at mid-tide level was composed of fine sand with an average median diameter of 0.25 mm ranging from 0.16 mm to 0.40 mm. Fill from Hampton Harbor had a median diameter of 0.37 mm to 0.56 mm, and from Hampton River it had an average median diameter of 0.34 mm ranging from 0.17 mm to 0.61 mm, all predominantly fine sand, but coarser than the original beach.

Comparison of beach slopes before and after fill placement by means of their steepness characteristics is shown in Table 2. The comparison shows that the fill, placed initially at a steeper slope than the original beach, has tended to assume the original slope.

[2] "Coast Erosion and the Development of Beach Profiles," Tech. Memo, No. 44, Beach Erosion Bd., Corps of Engrs., June, 1954.

[3] Hampton Beach, N. H., Beach Erosion Control Study, House Document No. 325, 83rd Congress, 2nd Session, 1954.

Beach profiles were surveyed in June, 1955 before filling and in December 1955 and January 1959 after filling. Profiles were run across the beach and out to a depth 18 ft below mean low water. Their layout is shown on Fig. 3. Changes in the high water shore line position along the profiles are shown graphically on Fig. 4. A large recession of the mean high water line of up to 200 ft occurred up to January 1959 between profiles 1 and 10 and a smaller one from profiles 19 and 26 while accretion moved the shore seaward up to 50 ft between profiles 11 and 18.

Volumetric changes are shown in Table 3 tabulated separately for the beach zones above and below mean low water. The only significant losses of fill occurred north of profile 10 above low water and north of profile 4 below low water and they amounted to 107,000 cu yd. The maximum annual rate of loss occurred at profile 3. Volumetric gains occurred at almost all other

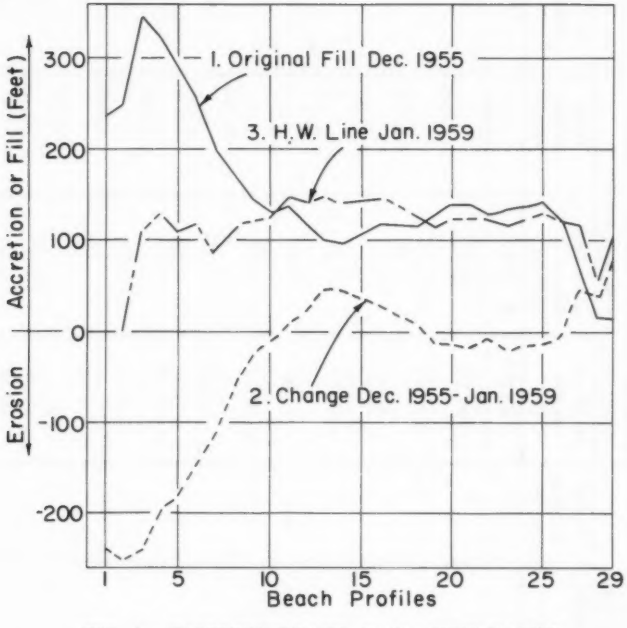


FIG. 4.—HAMPTON BEACH, N. H.—HIGH WATER SHORE LINE CHANGES

profiles. The rate of loss estimated prior to filling based on past beach changes had been 22,700 cu yd per yr,[3] which, surprisingly, corresponds rather closely to annual fill losses of approximately 20,000 cu yd measured only above low water. The large gain in the offshore area was unexpected. The coarseness and hence lack of any appreciable prior change along the north end of the original beach had made it impossible to accurately estimate losses of sand fill from this area.

Plum Island, Newburyport and Newbury, Mass.—The shore of about 4,000 ft of Plum Island was widened by placement of 560,000 cu yd of hydraulic sand fill during June and July 1953 obtained from "The Basin," an arm of the

TABLE 3.—VOLUMETRIC CHANGES, HAMPTON BEACH, N. H.

Profile	Original Fill December 1955			Volumetric Changes December 1955 to January 1959					
				Above Low Water			Low Water to 18' Depth		
	Volume, cu yds per linear ft	Length Shore, ft	Total Volume, cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cubic yards	cu yd per linear ft	cu yd per linear ft per yr	Total cubic yards
1				- 82.78[a]	- 26.10		0[b]	0[b]	
		200				-18,096			- 1,200
2				-100.19	- 31.63		- 12.00	- 3.79	
		200				-20,166			- 5,299
3				-101.48	- 32.04		- 17.70	- 5.59	
		200				-18,321			- 2,307
4				- 81.74	- 25.81		17.22	5.44	
		200				-15,185			9,837
5				- 70.19	- 22.16		87.55	27.64	
		200				- 9,177			17,866
6				- 21.67	- 6.84		91.19	28.79	
		200				- 5,974			17,527
7				- 38.07	- 12.02		78.81	24.88	
		200				- 6,140			17,942
8				- 23.33	- 7.37		100.59	31.76	
		200				- 4,251			21,921
9				- 19.19	- 6.06		122.33	38.62	
		200				- 1,363			22,408
10				5.56	1.76		100.07	31.59	
		200				1,389			17,378
11				7.78	2.46		73.70	23.27	
		200				1,732			16,153
12				9.44	2.98		87.85	27.73	
		200				3,667			18,980
13				27.22	8.59		101.96	32.19	
		200				5,001			19,180
14				27.04	8.54		89.82	28.36	
		200				4,962			16,791
15				22.59	7.13		78.07	24.65	

16		200		26.30	8.30	4,889	68.00	21.47	14,607
17		200		- 8.33	- 2.63	1,796	57.89	18.28	12,589
18		200		13.89	4.39	555	56.85	17.95	11,475
19		200		0	0	1,389	55.81	17.62	11,266
20		200		- 3.15	- 0.99	- 315	56.81	17.93	11,263
21		200		3.33	1.05	- 19	57.78	18.24	11,459
22		200		3.33	1.05	667	37.85	11.95	9,563
23		200		- 7.22	- 2.28	- 371	17.89	5.64	5,574
24		200		2.04	0.64	- 518	60.74	19.18	7,861
25		200		8.70	2.75	1,074	103.52	32.68	16,423
26		200		8.15	2.57	1,685	116.78	36.87	22,029
27		200		16.30	5.15	3,075	130.00	41.04	24,678
28		188		19.26	6.08	3,555	126.30	39.87	25,632
29				17.04	5.38	3,410	122.59	38.70	23,394
Total	57.4[c]	-	350,000[d]	- 10.0	- 3.2	-61,050	64.8[c]	20.5[c]	394,990

[a] - denotes erosion. No sign denotes accretion
[b] Assumed
[c] Based on scaled shore length of 6,100 ft
[d] Approximate volume. Another 47,000 cu yd placed north of profile 1

Totals Above and Below Low Water:
Accretion of 333,940 cu yd
= 54.8 cu yd per linear ft
= 17.3 cu yd per linear ft per yr

Merrimack River behind the beach (Fig. 5). The filling was necessitated by erosion of the beach and dunes with consequent damage to and loss of cottages and the danger that continuation of the erosion would breach the island[4] opposite "The Basin." The shore is directly exposed to wave attack from the Atlantic Ocean. The movement of littoral drift in the offshore area is generally from north to south. The direction of littoral drift along the beach, however, is variable due to the existence of shifting sand bars at the river mouth and opposite the beach.

The original beach consisted of medium sand at mid-tide level with a median diameter of 0.57 mm, ranging from 0.42 mm to 0.73 mm. Samples of the fill in place were slightly coarser with a median diameter of 0.64 mm ranging from 0.48 mm to 0.78 mm.

Comparison of beach slopes before and after filling by means of their steepness characteristics is shown in Table 4. The comparison shows that the fill was generally placed on a steeper slope than the original beach, but subsequently assumed a much flatter slope.

Surveys of beach profiles, shown in Fig. 5, were made before and after filling in June and July 1953, and at varying time intervals thereafter. They

TABLE 4.—COMPARISON OF STEEPNESS CHARACTERISTICS, PLUM ISLAND

Profile Date	Steepness Characteristics		
	Before Fill June 1953	After Fill July 1953	After Fill October 1955
4S	.038	.060	.017
4N	.063	.082	.057
12N	.077	.083	.061
20N	.080	.056	.037
Average	.064	.070	.043

did not cover all the fill area which extended from profile 10 south to profile 30 north. Changes of the high water shore line positions are shown in Fig. 6. The surveys of December 1953, September 1954, and October 1955 showed a high water line recession continuously along most of the beach with the greatest recession shifting successively southward from profile 8 north to 6 south. The net effect was a maximum loss of about 230 ft at profile 4 south, and lesser losses elsewhere, except at profile 10 south where a small gain occurred. Incomplete surveys of 1956 and 1958 show some gain of beach width in the area of greatest prior loss, probably due to construction of a groin near profile 2 south.

Volumemetric changes are included in Table 5. The results of changes from the incomplete survey of December 1953 are tabulated only to permit comparison of individual profiles. Heavy net losses of fill occurred from the beach as a whole up to October 1955, although some sections experienced gains. Total computed losses were 331,780 cu yd or approximately 59% of the

[4] Plum Island, Mass., Beach Erosion Control Study, House Document No. 243, 83rd Congress, 2nd Session, 1953.

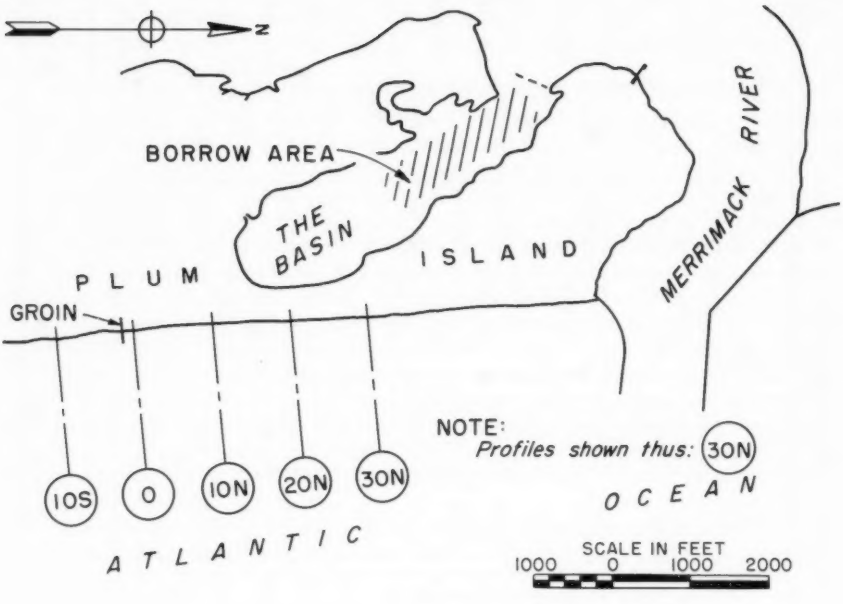


FIG. 5.—PLUM ISLAND, MASS.

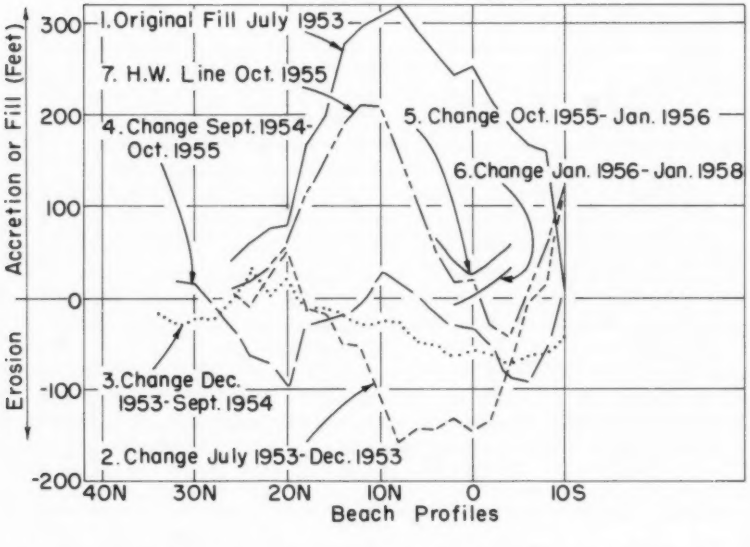


FIG. 6.—PLUM ISLAND, MASS.—HIGH WATER SHORE LINE CHANGES

TABLE 5.—VOLUMETRIC CHANGES PLUM ISLAND, MASSACHUSETTS

Profile	Length Shore, feet	Volumetric Changes July 1953 - Dec 1953			Volumetric Changes July 1953 - Sept 1954			Volumetric Changes Sept 1954 - Oct 1955			Volumetric Changes July 1953 - Oct 1955		
		cu yd per linear ft	cu yd per linear ft per yr	Total cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd
10S		52.04	125.0		- 85.18	- 73.0[b]		127.31	117.5		42.13	18.05	
	200			7,260						9,377			-85,556
8S		20.56	49.3				-74,415	-33.52	-30.9				
	400									-20,518			
4S					-162.87	-139.7		-69.05	-63.7		-231.92	-99.38	
	400						-69,093			-21,981			-91,074
0				--	-182.59	-157.0		-40.83	-37.7		-223.42	-95.74	
	400						-67,852			-29,388			-97,240
4N					-156.67	-134.2		-106.11	-97.9		-262.78	-112.60	
	400						-63,351			-17,425			-80,776
8N		-160.56	-385.0		-160.09	-137.1		+18.98	17.5		-141.11	-60.47	
	400						-29,518			+13,425			-16,093
12N				--	12.50	10.7		+45.15	41.7		57.65	24.70	
	400						13,550			+25,647			39,197
16N		9.54	22.9		55.28	47.4		+80.09	73.9		135.37	58.00	
	200			1,611			6,574			+5,509			12,083
18N		6.57	15.8		10.46	9.0		-25.00	-23.1		-14.54	-6.23	
	20			4,611			3,243			-10,157			-6,914
20N		39.54	9.48		21.94	18.8		-76.57	-70.7		-54.63	-23.41	
	400			4,583			4,481			-9,888			-5,407
24N		6.30	15.1		0.46	0.4		+27.13	25.0		27.59	11.78	
Total	3,400	--	--	--	- 81.6	- 69.8	-276,381	-16.3	-15.0	-55,399	- 97.5	- 43.3	-331,780

[b] - Denotes erosion. No sign denotes accretion.

original fill. Maximum annual rates of change varied widely on individual profiles. It has been estimated, based on prior changes of the beach,[4] that annual losses would be 36,000 cu yd per year, approximately one-fourth of losses actually experienced:

Misquamicut Beach, Westerly, R. I.—The behavior of fill placed on this beach is of particular interest because it illustrates the rapidity with which fill changes can occur. Misquamicut Beach on the south shore of Rhode Island is directly exposed to the Atlantic Ocean to the south. The project consisted of straightening an indented shore line for protection of the state beach development and for recreational use.[5] Original beach material consisted of fine sand. Fill obtained by excavation in a lagoon behind the beach was so much coarser that it was placed beneath the original beach material. About 3,200 ft of shore were widened, generally from 60 ft to 100 ft, by placement of 80,000 cu yd of fill up to July, 1959. A series of fall storms in October and November 1959 resulted in rapid erosion of the fill. Surveys were run in November 1959 out to the low water line repeating prior surveys and they showed that practically all of the fill was lost and the high water shore line had receded to the position it occupied before fill placement. It was assumed that the fill was also lost in the zone below low water. Visual inspection of the beach in June 1960 showed that it had recovered a small amount of material

TABLE 6.—COMPARISON OF STEEPNESS CHARACTERISTICS, WHITE SAND BEACH

Profile	Steepness Characteristics		
	Before Fill July 1956	After Fill May 1957	After Fill May 1959
2	.043	.037	.041
8	.034	.048	.022
14	.043	.032	.030
Average	.040	.039	.031

but it was still indented. A considerable amount of accretion was observed to have occurred beyond the east limit of the state property attributed to loss of the fill from the state beach. It had been estimated, based on past changes, that beach fill would be lost at a rate of 4,000 cu yd per yr whereas actual losses for a short period were at a rate of approximately 240,000 cu yd per yr. Subsequent gains show that this high rate of loss did not continue.

White Sand Beach, Old Lyme, Conn.—The easterly 1,360 ft of White Sand Beach was widened by placement of approximately 50,000 cu yd of sand fill from an inland borrow area during December 1956 and January 1957. The fill was placed to restore and improve a public and private beach and to provide storm protection for a shore cottage development.[6] An impermeable stone groin was built at the west end of the fill to retain it and to reduce losses from littoral drift which is predominantly westward. The shore faces Long Island Sound. Waves are of short period generated over limited fetches. Two

[5] South Kingstown and Westerly, R. I., Beach Erosion Control Study, House Document No. 30, 86th Congress, First Session, 1958.

[6] Area 6—Niantic Bay to Connecticut River, Conn., Beach Erosion Control Study, House Document No. 84, 83rd Congress, First Session, 1953.

samples of the original beach at mid-tide elevation had median diameters of 0.86 mm and 1.30 mm with high percentages of gravel, medium, and coarse sand. The fill was much finer consisting of sand with median diameters varying from 0.25 mm to 1.10 mm, averaging 0.46 mm.

Comparison of slopes before and after fill placement by means of their steepness characteristics is made in Table 6. The comparison indicates that the fill in May 1957, a few months after placement, had assumed a slope slightly flatter than the original beach and in the next 2 yr it took a still flatter slope. This flattening of slope was accompanied by shoaling in the flat part of the beach profile in the vicinity of the low water line.

Surveys of beach profiles shown on Fig. 7, were made on July 1956, before filling, and on May 1957, March 1958, April 1958, and May 1959 after filling. Changes of the mean high water shore line positions are shown graphically in Fig. 8. The high water line receded landward almost continuously along all but the west end of the beach adjacent to the groin. As a result of all changes up to May 1959, the high water shore line moved landward over 100 ft to its position before filling at the east end of the beach, and gradually lesser amounts towards the west end, while immediately adjacent to the groin, accretion or landward movement of about 25 ft occurred.

Volumetric changes are included in Table 7. It is interesting to note that 30% of all losses occurred as a result of one storm period between March and April 1958. Decreasing rates of loss during the latter period, April 1958 to March 1959, are probably due in part to almost complete loss of fill from the east end of the beach which had been subject to the most rapid erosion. The average annual rate of loss experienced for the entire beach was 9,400 cu yd which was higher than the rate of 2,300 cu yd estimated prior to fill placement.[6]

West Silver Sands Beach, East Haven, Conn.—West Silver Sands Beach is a low sandy barrier bar with a history of flooding, storm damages, and shore recession. The beach faces and is exposed to short-period waves from limited fetches across Long Island Sound. Restoration, improvement, and protection of the beach was undertaken by placement of about 158,000 cu yd of sand fill dredged from offshore during June to August 1957 (Fig. 9). An impermeable stone groin was constructed to retain the fill at a creek at its east limit. Two samples of original beach material from mid-tide level consisted predominantly of medium sand with median diameters of 0.66 mm and 1.00 mm. Seven borings from the offshore borrow area had an average median diameter of 0.54 mm, ranging between 0.26 mm and 1.04 mm. The borings were finer than the original beach, but due to probable loss of fines during hydraulic placement, this may not have been true of the fill in place.

Comparison of slopes before and after fill placement by means of their steepness characteristics is shown in Table 8. The comparison indicates that the fill was initially placed on a much flatter slope than the original beach and has maintained this flatter slope.

Surveys of beach profiles shown in Fig. 9 were run in June 1957 before filling and in October 1957, June 1958, and April 1959 after filling. Changes of the mean high water line positions are shown graphically on Fig. 10. Changes consisted of accretion and seaward movement of up to about 25 ft in the central portion of the beach and recession of about 36 ft at the east and west ends. The recession along the east end occurred up to June 1958 and along the west end in the following period to April 1959.

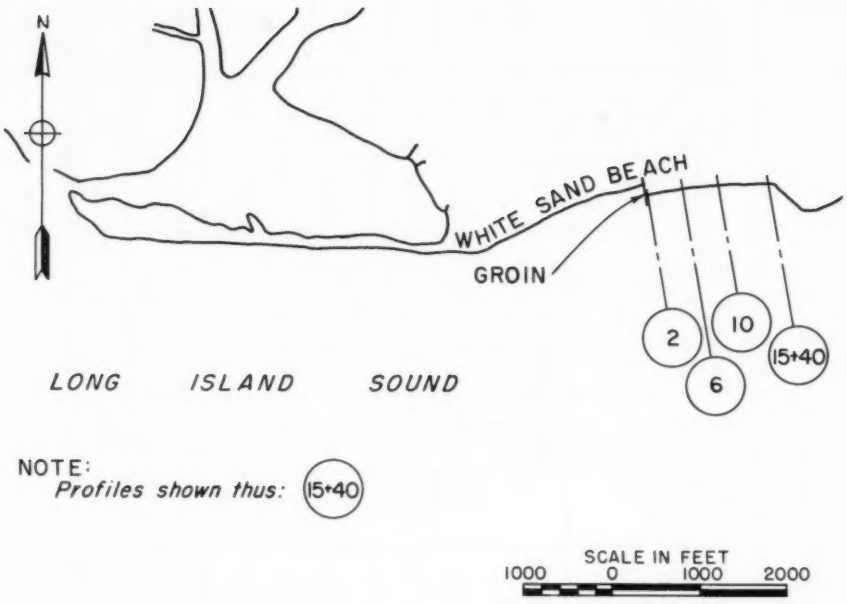


FIG. 7.—WHITE SAND BEACH, CONN.

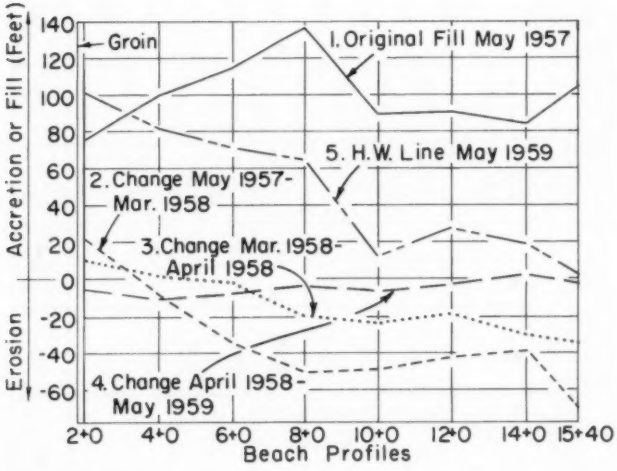


FIG. 8.—WHITE SAND BEACH, CONN.—HIGH WATER SHORE LINE CHANGES

TABLE 7.—VOLUMETRIC CHANGES WHITE SAND BEACH, CONN.

Profile	Original Fill May 1957			Volumetric Changes May 1957 - March 1958			Volumetric Changes March 1958 - April 1958			Volumetric Changes April 1958 - May 1959			Volumetric Changes May 1957 - May 1959		
	Volume, cu yd per linear ft	Length Shore, ft	Total Volume, cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd
Groin	27.13			8.24	9.96		2.59	+27.01		-3.70	-3.51		+ 7.13	+ 3.61	
		20	542			165			52			- 74			143
2	27.13			8.24	9.96		2.59	+27.01		-3.70	-3.51		+ 7.13	+ 3.61	
		200	6,342			898			102			-750			250
4	36.30			0.74	0.89		-1.57	-16.37		-3.79	-3.60		- 4.62	- 2.34	
		200	7,380			- 676			- 222			-750			-1,648
6	37.50			- 7.50[a]	- 9.07		-0.65	- 6.78		-3.70	-3.51		-11.85	- 6.00	
		200	7,907			-1,769			- 639			-426			-2,834
8	41.57			-10.18	-12.31		-5.74	-59.81		-0.56	-0.53		-16.48	- 8.34	
		200	6,750			-2,621			- 944			-121			-3,686
10	25.93			-16.01	-19.36		-3.70	-38.58		-0.65	-0.62		-20.36	-10.31	
		200	5,259			-2,685			-1,128			+ 74			-3,739
12	26.67			-10.83	-13.10		-7.59	-79.15		+1.39	+1.32		-17.03	- 8.62	
		200	5,065			-2,077			-1,602			+ 18			-3,661
14	23.98			- 9.62	-11.63		-8.43	-87.91		-1.20	-1.14		-19.25	- 9.75	
		140	4,582			-2,320			-1,231			- 85			-3,636
15+40	41.48			-23.51	-28.43		-9.17	-95.63		0	0		-32.68	-16.55	
Total	32.2	1,360	43,827	- 8.15	- 9.85	-11,085	-4.13	-43.02	-5,612	-1.55	-1.47	-2,114	-13.83	- 7.00	-18,811

[a] - denotes erosion. No sign denotes accretion

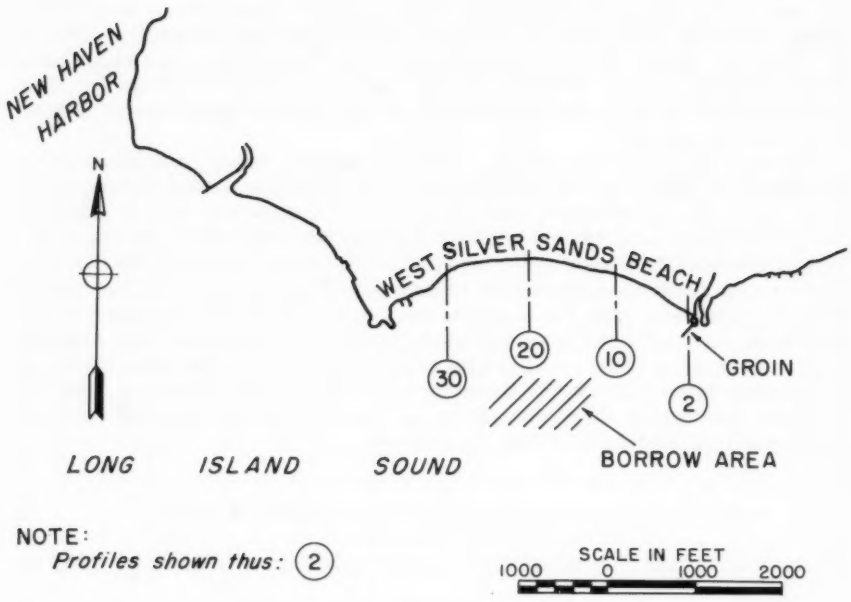


FIG. 9.—WEST SILVER SANDS BEACH, CONN.

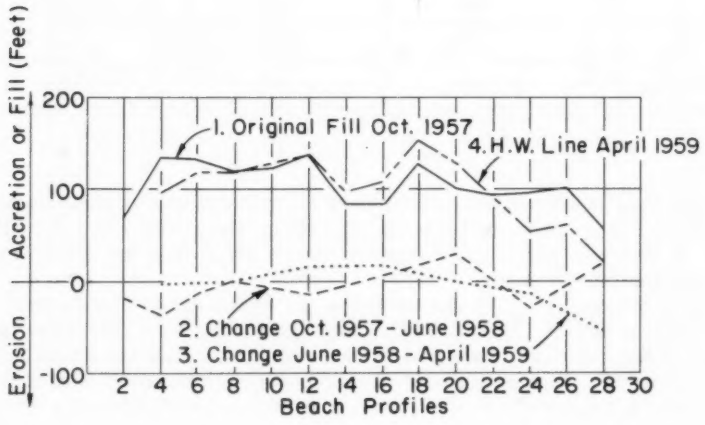

Accretion or Fill (Feet)
200
100
0
-100
Erosion
1. Original Fill Oct. 1957
4. H.W. Line April 1959
2. Change Oct. 1957 - June 1958
3. Change June 1958 - April 1959
2 4 6 8 10 12 14 16 18 20 22 24 26 28 30
Beach Profiles

FIG. 10.—WEST SILVER SANDS BEACH, CONN.—HIGH WATER SHORE LINE CHANGES

Volumetric changes are shown in Table 9. During the period October 1957 to June 1958, losses occurred between profiles 2 and 12 and accretion along the rest of the beach, but during the ensuing period, June 1958 to April 1959, losses occurred only along the west end of the beach between profiles 24 and 28. The net result of all volume changes was accretion on the beach as a whole although losses occurred at the ends amounting to 6,065 cu yd. The loss of fill from the entire beach based on past behavior had been estimated at 2,900 cu yd per yr.[7]

Prospect Beach, West Haven, Conn.—Prospect Beach was widened by placement of approximately 443,000 cu yd of hydraulic sand fill along 6,470 ft of shore during February 1957 (Fig. 11). The work was done to improve the public beach and protect a shore highway and residential development. Eight impermeable groins were constructed to reduce losses of the fill. The shore faces Long Island Sound and is exposed to wave attack from the southwest to northeast with the longest fetch and most severe exposure to the southwest. Littoral drift moves predominantly to the northeast. The original beach was coarse being covered with gravel and cobbles. Two samples taken at mid-tide level had median diameters of 6.70 mm and 54.00 mm. The fill in place was much finer consisting of fine and medium sand with median diameters ranging from 0.22 mm to 0.89 mm.

TABLE 8.—COMPARISON OF STEEPNESS CHARACTERISTICS, WEST SILVER SANDS BEACH

Profile	Steepness Characteristics		
	Before Fill June 1957	After Fill October 1957	After Fill April 1959
4	.061	.024	.025
8	.064	.017	.028
16	.061	.037	.047
24	.053	.062	.051
Average	.060	.035	.038

Comparison of slopes before and after filling by means of their steepness characteristics is shown in Table 10. The comparison shows that the fill was placed at and generally maintained a flatter slope than the original beach though steepening did occur.

Surveys of beach profiles shown on Fig. 11 were repeated on January 1956 before filling and on March 1957, June 1958, and June 1959 after filling. Changes of the positions of the mean high water shore lines are shown graphically in Fig. 12. Changes consisted of recession at the south end of the fill and accretion adjacent to it as a result of northward migration of a stockpile of sand placed here, irregular small changes mostly recession along the middle of the beach, and accretion at the north end.

[7] Area 9—East River to New Haven Harbor, Conn., Beach Erosion Control Study, House Document No. 395, 84th Congress, 2nd Session, 1957.

TABLE 9.—VOLUMETRIC CHANGES WEST SILVER SANDS BEACH, CONN.

Profile	Original Fill October 1957			Volumetric Changes Oct 1957 - June 1958			Volumetric Changes June 1958 - April 1959			Volumetric Changes Oct 1957 - April 1959		
	Volume, cu yd per linear ft	Length Shore, ft	Total Volume, cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd
		30	620			- 96						
2	41.30			- 7.13[a]	-10.7				776			-1,143
		200	11,274			-1,823						
4	71.48			-11.11	-16.7		6.75	8.1		-4.36	-2.9	
		200	15,482			-2,241						-2,529
6	83.33			-11.30	-16.9				1,536			
		200	17,178			-1,824						
8	88.44			- 7.13	-10.7		0.93			-6.20	-4.1	
		200	17,512									852
10	86.67					-1,536			2,388			
		200	15,074									
12	64.07			- 0.56	- 0.8		11.02	1.3		+10.46	7.0	
		200	11,704									4,074
14	52.96					+ 259			3,815			
		200	10,126									
16	48.33			+ 1.85	2.8		8.05	9.7		+ 9.90	6.6	
		200	10,570									4,715
18	57.41					+ 2,114			2,601			
		200	11,466									
20	57.26			+ 8.72	13.1		1.76	2.1		+10.48	7.0	
		200	9,614									3,143
22	38.89					+ 3,541			- 398			
		200	8,585									
24	46.96			+ 8.98	13.5		-3.70	-4.4		+ 5.28	3.5	
		200	10,238									- 889
26	55.41					+ 2,277			-3,166			
		200	5,466									
28	19.30			+ 2.41	3.6		-12.13	-14.6		-9.72	-6.5	
		60	580			+ 72			-1,576			-1,504
30	0			0						0	0	
Total	58.7	2,690	157,940	+ 0.28	0.4	+743	+ 2.22	+ 2.7	+ 5,976	2.50	1.7	+ 6,719

[a] - denotes erosion. No sign denotes accretion.

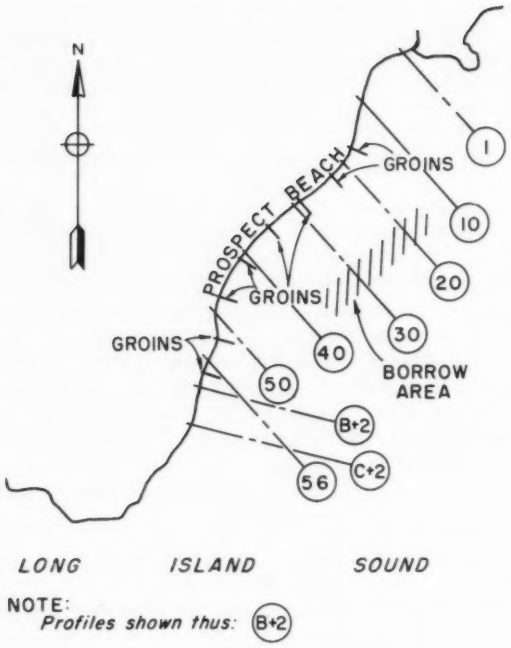


FIG. 11.—PROSPECT BEACH, CONN.

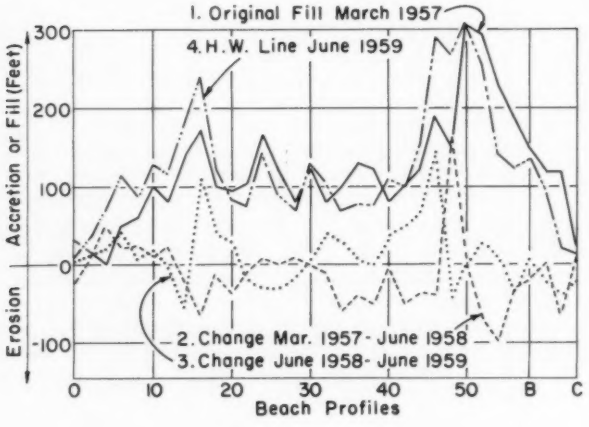


FIG. 12.—PROSPECT BEACH, CONN.—HIGH WATER SHORE LINE CHANGES

Volumetric changes are shown in Table 11. Almost all fill losses occurred south of profile 52 while gains exceeding the losses occurred to the north. The net effect for the beach as a whole was a gain of 39,526 cu yd, an annual rate of 17,500 cu yd. It has been estimated[8] that fill losses for the entire beach would be 8,400 cu yd per yr.

Woodmont Shore, Milford, Conn.—Two segments of Woodmont Shore, 500 and 3,800 ft long, located west and north of Merwin Point, respectively, were widened by placement of about 256,000 cu yd of hydraulic fill between November 1958 and April 1959 (Fig. 13). The work was done to improve public beach areas and to protect a shore road and residential property. Five groins were constructed to reduce losses of the fill from the predominantly northward littoral drift. The shore is exposed to wave attack across Long Island Sound from the southwest to the northeast, the most severe exposure being from the longer fetch to the southwest. The original shore was largely covered with gravel and cobbles, with sand only in indentations along short segments. The fill was much finer consisting of fine to medium sand. Borings from the borrow area had median diameters of 0.24 mm and 0.31 mm.

TABLE 10.—COMPARISON OF STEEPNESS CHARACTERISTICS PROSPECT BEACH

Profile	Steepness Characteristics		
	Before Fill January 1956	After Fill March 1957	After Fill June 1959
10	.015	.011	.010
20	.034	.021	.022
32	.039	.024	.037
42	.036	.024	.011
B+2	.021	.019	.030
Average	.029	.020	.022

Comparison of slopes before and after fill placement by means of their steepness characteristics is made in Table 12. The comparison shows that the fill was placed on a much flatter slope than the original beach and has maintained this slope. The surveys of beach profiles show that a large amount of shoaling occurred for a considerable distance seaward of the low water line.

Surveys of beach profiles shown on Fig. 13 were repeated in March 1958 before filling, and in April 1959 and January 1960 after filling. Changes of the position of the mean high water line are shown on Fig. 14. The high water line receded along all but short segments of the north end of the beach where the terminal groin resulted in some accretion by impounding littoral drift. In several places the shore receded to the position it occupied before filling.

Volumetric changes are shown in Table 13. The survey of January 1960 did not repeat all the profiles. Volumetric change computations are subject

[8] Area 3—New Haven Harbor to Housatonic River, Conn., Beach Erosion Control Study, House Document No. 203, 83rd Congress, First Session, 1953.

TABLE 11.—VOLUMETRIC CHANGES PROSPECT BEACH, CONN.

Profile	Original Fill March 1957			Volumetric Changes March 1957 - June 1958			Volumetric Changes June 1958 - June 1959			Volumetric Changes March 1957 - June 1959		
	Volume, cu yd per linear ft	Length Shore, ft	Total Volume, cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd
0	9.62			2.04	1.6		- 1.30	- 1.30		0.74	0.33	
		200	2,111			630			185			815
2	11.48			4.26	3.4		3.15	3.15		7.41	3.29	
		200	4,537			1,500			630			2,130
4	33.89			10.74	8.6		3.15	3.15		13.89	6.17	
		200	7,907			1,481			1,092			2,573
6	45.19			4.07	3.3		7.78	7.78		11.85	5.27	
		194.5	9,563			1,315			558			1,873
8	53.15			9.44	7.6		- 2.04	- 2.04		7.40	3.29	
		164.1	8,585			2,772			- 896			1,876
10	51.48			24.26	19.4		- 8.89	- 8.89		15.37	6.83	
		164.1	8,887			3,039			- 456			2,583
12	56.85			12.78	10.2		3.34	3.34		16.12	7.16	
		160.8	9,886			1,534			357			1,891
14	66.11			6.30	5.0		1.11	1.11		7.41	3.29	
		155.5	10,971			590			1,037			1,627
16	75.00			1.30	1.0		12.22	12.22		13.52	6.01	
		155.5	9,992			979			1,814			2,793
18	53.51			11.30	9.0		11.11	11.11		22.41	9.96	
		184.2	9,295			2,029			716			2,745
20	47.40			10.37	8.3		- 3.33	- 3.33		7.04	3.13	
		189.5	11,475			702			-1,439			- 737
22	73.70			- 3.06[d]	- 2.4		-11.85	-11.85		-14.91	- 6.63	
		199.3	15,335			701			- 166			535
24	80.18			10.00	8.0		10.18	10.18		22.18	9.86	
		199.3	14,025			1,698			351			2,049
26	60.56			7.04	5.6		- 6.67	- 6.67		0.37	0.16	
		199.3	11,644			2,620			-1,476			1,144
28	56.30			19.26	15.4		- 8.15	- 8.15		11.11	4.94	
		199.3	12,567			4,097			-1,845			2,252
30	69.81			21.85	17.5		-10.37	-10.37		11.48	5.10	

32	47.78	199.3	11,718	6.48	5.2	2,823	11.85	11.85	148	18.33	8.15	2,971
34	47.41	199.3	9,485	12.04	9.6	1,845	2.04	2.04	1,384	14.08	6.26	3,229
36	69.44	198.2	11,580	2.22	1.8	1,413	9.44	9.44	1,138	11.66	5.18	2,551
38	71.30	198.2	13,947	- 4.26	- 3.4	- 202	8.51	8.51	1,780	4.25	1.89	1,578
40	55.00	197.7	12,484	10.56	8.4	622	16.48	16.48	2,471	27.04	12.02	3,093
42	86.85	195.4	13,786	10.19	8.2	2,026	25.59	25.59	3,818	35.78	15.90	5,844
44	117.04	195.4	19,848	-13.15	-10.5	- 290	20.19	20.19	4,179	7.04	3.13	3,889
46	122.41	187.8	22,483	18.89	15.1	539	26.11	26.11	4,347	45.0	20.0	4,886
48	131.48	163.0	20,691	19.07	15.3	3,094	21.67	21.67	3,894	40.74	18.11	6,988
50	163.15	163.0	24,012	11.50	9.2	2,249	14.81	14.81	2,973	26.31	11.69	5,222
52	167.59	175.6	29,055	-22.41	-17.9	-1,219	3.33	3.33	1,593	-19.08	- 8.48	374
54	144.07	175.9	27,411	-31.67	-25.3	-6,182	11.67	11.67	1,319	-20.0	- 8.89	-4,863
56	119.63	175.9	23,193	- 9.26	- 7.4	-3.599	- 9.07	- 9.07	228	- 0.19	- 0.08	-3,371
B	115.56	-	13,064	- 3.89	- 3.1	- 526	- 6.11	- 6.11	- 607	- 2.22	- 0.99	-1,133
B 2	60.93	200	17,648	- 3.33	- 2.7	- 722	-23.15	-23.15	-2,926	-26.48	-11.77	-3,648
B 4	87.22	200	14,815	-26.30	-21.0	-2,963	-21.30	-21.30	-4,444	-47.60	-21.16	-7,407
C	22.41	178	10,963	- 1.30	- 1.0	-2,456	-13.15	-13.15	-3,444	-14.45	- 6.42	-5,900
C 2		200		5.19	4.2	389	0	0	-1,315	5.19	2.31	- 926
Total	68.5[a]	6,028[b]	442,963	3.38[c]	2.7[c]	22,528	2.55[c]	2.55[c]	16,998	5.93[c]	2.64[c]	39,526

[a] Based on scaled shore length of 6,470 ft. [b] Approximate distance between profile stations excluding distance from 56 to B. [c] Based on scaled shore length of 6,670 ft. [d] - denotes erosion. No sign denotes accretion.

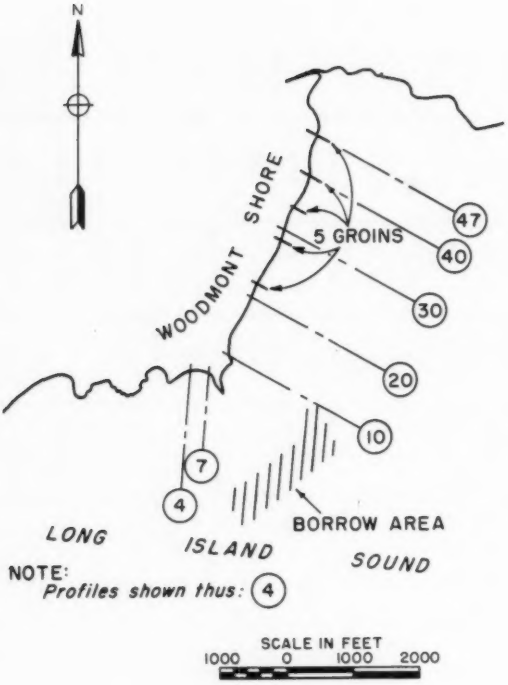


FIG. 13.—WOODMONT SHORE, CONN.

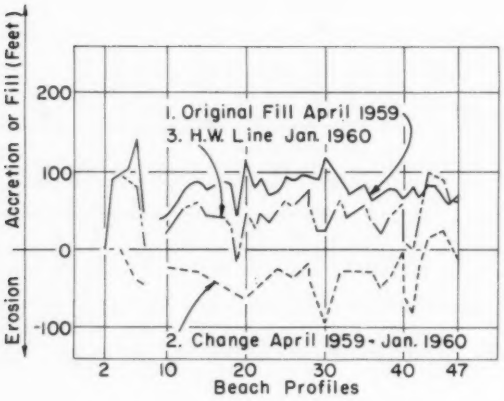


FIG. 14.—WOODMONT SHORE, CONN.—HIGH WATER SHORE LINE CHANGES

to some error due to the wide spacing of profiles surveyed, particularly where profiles straddle groins. The effect of the groins in holding fill on their south sides is evident at profiles 28S and 40S where the surveys were run on both sides of the groins. Losses of fill occurred along all but the north end of the beach. Approximately 40,000 cu yd of the fill was lost, an annual rate of over 53,000 cu yd. It had been estimated[8] that the loss of fill would be 6,700 cu yd per yr.

Gulf Beach, Milford, Conn.—Gulf Beach was widened by direct placement of hydraulic fill along 1,235 ft of shore during April and May 1957 (Fig. 15). The fill was placed to improve and enlarge a public bathing beach for recreational use. The beach was located south of and adjacent to a stone jetty which already held a smaller sandy beach on its south side. The shore is exposed to wave attack across Long Island Sound from the south and west. Due to the orientation of the shore, littoral drift moves predominantly northward. Except

TABLE 12.—COMPARISON OF STEEPNESS CHARACTERISTICS WOODMONT SHORE

Profile	Steepness Characteristics		
	Before Fill March 1958	After Fill April 1959	After Fill January 1960
14	.034	.021	.026
20	.034	.027	.020
24	.044	.026	.023
32	.037	.019	.022
36	.036	.021	.021
45	.042	.012	.012
Average	.038	.021	.021

for the area adjacent to the jetty, the original shore was covered with gravel and cobbles. A sample of the beach at mid-tide level had a median diameter of 6.90 mm. The fill was much finer. Eight samples in place had a median diameter of 1.04 mm ranging from 0.16 mm to 4.76 mm.

Comparison of the slopes before and after fill placement by means of their steepness characteristics is made in Table 14. The comparison shows that the fill was placed on a much flatter slope than the original beach and except at profile 2 kept a flatter slope.

Surveys of beach profiles shown on Fig. 15 were repeated on March 1956, before filling and on May 1957, September 1957, and May 1959 after filling. Changes of the position of the mean high and low water lines are shown on Fig. 16. Changes consisted of recession of both lines almost to their positions before filling at the south end of the beach at profile 2 and seaward movement at the north end due to impounding by the jetty of the fill which had drifted northward.

TABLE 13.—VOLUMETRIC CHANGES WOODMONT SHORE, CONNECTICUT

Profile	Original Fill April 1959			Volumetric Change April 1959 - Jan 1960		
	Volume, cu yd per linear ft	Length Shore, ft	Total Volume, cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd
Pocket Beach West of Merwin Point						
2	24.0					
		100	2,986			
3	35.7					
		100	7,056			
4	105.4			-10.2[d]	-13.6	
		100	10,194			-1,926
5	98.5					
		100	7,551	- 9.1	-12.1	
6	52.5					-1,343
		100	4,800	-17.8	-23.7	
7	43.5					
Subtotal	67.2	500	32,587	-10.9[a]	-14.5[a]	-3,269
Beach North of Merwin Point						
9	16.2					
		100	2,912	-11.0	-14.7	
10	42.0					
		100	3,796			
11	33.9					
		100	3,079			-4,296
12	27.7					
		100	3,357			
13	39.4					
		100	4,338	-10.5	-14.0	
14	47.3					
		100	4,981			
15	52.3					
		100	6,028			
16	68.2					
		100	5,991			-3,721
17	51.6					
		100	4,070			
18	29.8					
		100	2,110			
19	12.4					
		100	3,745	- 8.1	-10.8	
20	62.5					
		100	6,218			
21	61.9					
		100	6,713			
22 (Groin)	72.4					-3,320
		100	6,845			
23	64.5					
		100	5,810	- 8.5	-11.3	
24	51.7					
		100	7,037			-4,028
25	89.1					
		100	8,148	-31.8	-42.4	
26	73.9					
		100	6,366			-3,601
27	53.4					
		100	5,792	- 4.3	- 5.7	
28S	--					
		0				0
28 (Groin)	62.4					
		0		- 9.7	-12.7	
28N	--					-3,889
		100	5,213			
29	41.9					
		100	4,824	-29.2	-38.9	
30	54.6					
		100	5,352			-4,259
31	52.4					
		100	6,755	-13.4	-17.9	
32	82.7					
		109	9,416			
33 + 09	90.1					
		91	8,215			-4,220
34 (Groin)	90.5					
		100	8,273			
35	75.0					
		100	6,435	- 7.7	-10.3	
36	53.7					-1,278
		100	5,074	-17.9	-23.9	-1,823
37	47.8					-1,218
		100	5,194	-18.6	-24.8	- 301
38	56.1					
		100	5,463	- 5.7	- 7.6	
39	53.1					
		100	5,051	- 0.3	- 0.4	
40S	--					
		0				0
40 (Groin)	47.9					
		0		-22.5	-30.0	
40N	--					-2,254
		100	4,688	-22.6	-30.1	
41	44.9					-1,444
		100	6,333	- 6.3	- 8.4	- 374
42	81.8					
		100	8,306	- 1.2	- 1.6	
43	84.4					
		100	8,060			971
44	76.8					
		100	8,078	10.9	14.5	
45	84.7					
		100	7,843			1,917
46	72.1					
		100	7,764			
47	83.1			8.2	10.9	
Subtotal	58.9	3,800	223,670	-10.0[b]	-13.3[b]	-37,138
Total	59.7	4,300	256,257	-10.1[c]	-13.5[c]	-40,407

[a] Based on a shore length of 300 ft. [b] Based on a shore length of 3,700 ft. [c] Based on a shore length of 4,000 ft. [d] - denotes erosion. No sign denotes accretion.

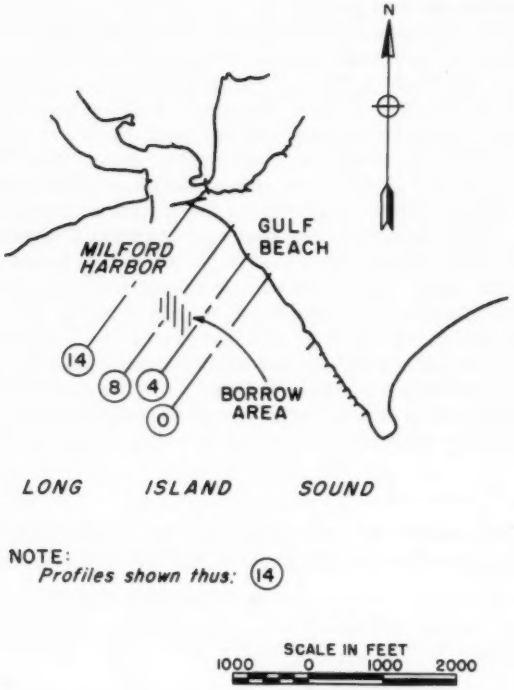


FIG. 15.—GULF BEACH, CONN.

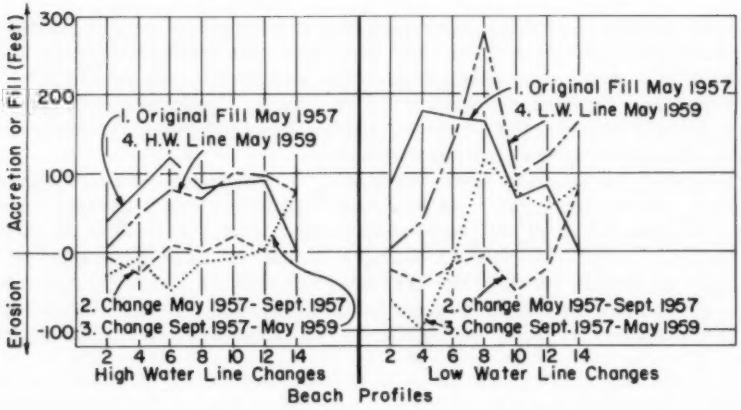


FIG. 16.—GULF BEACH, CONN.—HIGH AND LOW WATER SHORE LINE CHANGES

Volumetric changes are shown in Table 15. Losses of fill occurred along all the beach except north of profile 12, adjacent to the jetty. Annual rates of loss during the two periods of 4.4 and 4.0 cu yd per ft were fairly uniform. Actual losses were at an annual rate of approximately 5,000 cu yd which was higher than the rate of 1,100 cu yd estimated[8] prior to fill placement.

Seaside Park, Bridgeport, Conn.—The public bathing beach at Seaside Park was improved for recreational use and the park development was protected from recurring storm damages by placement of about 691,000 cu yds of hydraulic sand fill along 8,300 ft of shore during February and March 1957 (Fig. 17). The shore faces Long Island Sound and is exposed over limited fetches to wave attack from the northeast to southwest. There is no predominant direction of movement of littoral drift. The original beach was composed of fine to medium sand. Two beach samples from mid-tide level had median diameters of 0.24 mm and 0.55 mm. Eighty samples of the fill in place had an average median diameter of 0.37 mm ranging from 0.12 mm to 1.10 mm.

Comparison of the slopes before and after fill placement by means of their steepness characteristics is made in Table 16. The comparison shows that the fill was placed on a much flatter slope than the original beach and has assumed a still flatter slope along the easterly end of the beach.

TABLE 14.—COMPARISON OF STEEPNESS CHARACTERISTICS, GULF BEACH

Profile	Steepness Characteristics		
	Before Fill March 1956	After Fill May 1957	After Fill May 1959
2	.034	.023	.038
8	.035	.018	.016
12	.034	.019	.025
Average	.034	.020	.026

Surveys of beach profiles were repeated in March 1956 before filling and in March 1957, September 1957, and June 1959 after filling. Changes in the location of the mean high water lines are shown graphically on Fig. 18. The high water line receded along most of its length. The largest movement of 150 ft at profile 76 occurred where the fill had been placed to a greater width than elsewhere, forming a shore projection.

Volumetric changes are shown in Table 17. The beach as a whole gained in volume during the first 6 months after filling but lost a greater amount during the next 1-3/4 yr, the net result being a loss of 58,732 cu yd, mostly from the beach west of profile 70. Smaller losses occurred along most of the beach east of profile 50 while the area between profiles 50 and 70 had a substantial gain. Total net losses averages 26,000 cu yd per yr which was higher than the estimate of loss of 10,600 cu yd made prior to fill placement.[9]

[9] Area 7—Housatonic River to Ash Creek, Conn., Beach Erosion Control Study, House Document No. 248, 83rd Congress, 2nd Session, 1953.

TABLE 15.—VOLUMETRIC CHANGES GULF BEACH, CONN.

Profile	Original Fill May 1957			Volumetric Changes May 1957 - Sept 1957			Volumetric Changes Sept 1957 - May 1959			Volumetric Changes May 1957 - May 1959		
	Volume, cu yd per linear ft	Shore Length, ft	Total Volume, cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total Volume, cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total Volume, cu yd
So. Limit	0			0			0					
		50	573			40			- 561			- 521
2	22.2			1.57	4.32		-22.5	-13.9		-20.9	-10.5	
		200	6,706			-695			-3,787			-4,481
4	44.1			-8.51[a]	-23.40		-15.4	- 9.5		-23.9	-12.0	
		200	9,157			-880			-3,164			-4,044
6	47.5			-0.28	- 0.77		-16.3	-10.0		-16.6	- 8.3	
		200	8,981			65			-2,547			-2,481
8	42.4			0.93	2.56		- 9.2	- 5.7		- 8.3	- 4.2	
		200	8,181			- 74			-1,026			-1,100
10	39.4			-1.67	- 4.59		- 1.0	- 0.6		- 2.7	- 1.4	
		200	8,806			-695			- 444			-1,139
12	48.6			-5.28	-14.50		- 3.4	- 2.1		- 8.7	- 4.4	
		185	4,459			239			3,389			3,627
No. Limit	0			7.68	21.10		37.3	22.9		45.0	22.6	
Total	37.9	1,235	46,863	-1.6	- 4.4	-2,000	- 6.5	- 4.0	-8,140	- 8.1	- 4.1	-10,140

[a] Denotes erosion. No sign denotes accretion.

TABLE 16.—COMPARISON OF STEEPNESS CHARACTERISTICS, SEASIDE PARK

Profile	Steepness Characteristics		
	Before Fill March 1956	After Fill March 1957	After Fill June 1959
6	.056	.017	.010
20	.039	.017	.012
36	.044	.018	.014
58	.036	.007	.008
76	.015	.010	.011
Average	.038	.014	.011

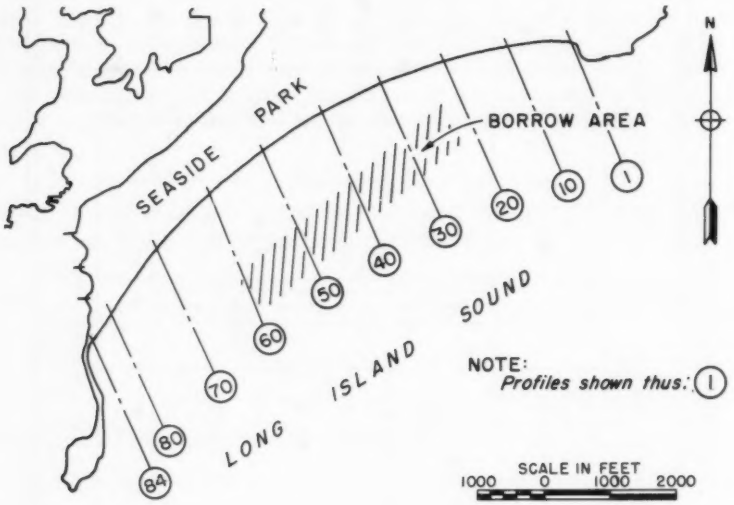


FIG. 17.—SEASIDE PARK, CONN.

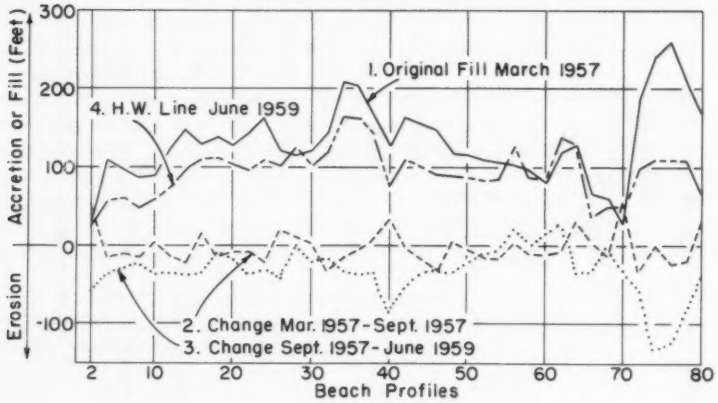


FIG. 18.—SEASIDE PARK, CONN.—HIGH WATER SHORE LINE CHANGES

Sherwood Island State Park, Westport, Conn.—Sherwood Island State Park was improved for recreational use during June 1957 by placement of about 557,000 cu yd of hydraulic fill along 5,950 ft of its shore (Fig. 19). A stone groin was built at the west end of the beach and a timber training wall at a

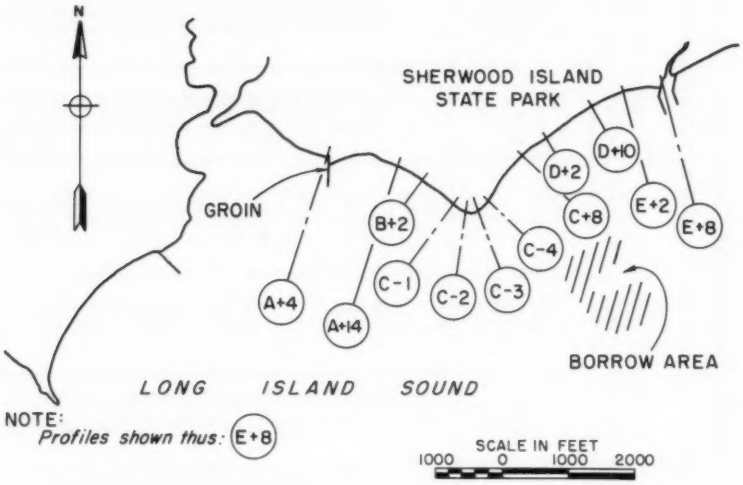


FIG. 19.—SHERWOOD ISLAND STATE PARK, CONN.

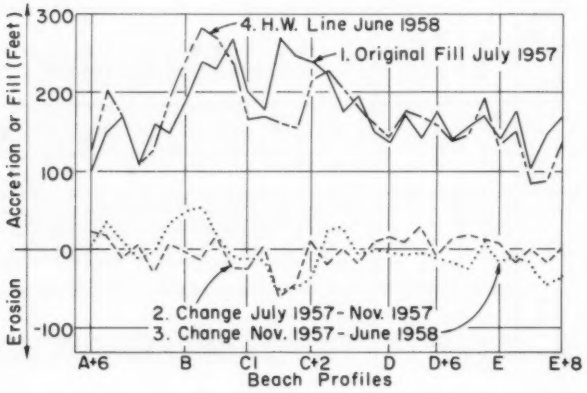


FIG. 20.—SHERWOOD ISLAND STATE PARK, CONN.—HIGH WATER SHORE LINE CHANGES

creek at the east limit to retain the fill. The beach is exposed to waves generated over limited fetches across Long Island Sound from southerly directions. Littoral drift moves northwestward, west of Sherwood Point. There is no known predominance east of Sherwood Point. The original beach was coarse

TABLE 17.—VOLUMETRIC CHANGES, SEASIDE PARK, CONNECTICUT

Profile	Original Fill March 1957			Volumetric Changes March 1957 - September 1957			Volumetric Changes September 1957 - June 1959			Volumetric Changes March 1957 - June 1959		
	Volume, cu yd per linear ft	Length Shore, ft	Total cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd
1	0											
		100	2,422									
2	48.40			13.70	27.4		- 8.79	- 5.0		4.91	2.18	
		200	10,770			1,648			-1,185			463
4	49.30			2.78	5.6		- 3.06	- 1.8		- 0.28	- 0.12	
		200	11,881			852			- 241			611
6	59.56			5.74	11.5		0.65	0.4		6.39	2.84	
		200	12,119			685			- 204			481
8	61.59			1.11	2.2		- 3.69	- 2.21		- 2.58	- 1.15	
		200	12,652			1,000			-1,639			- 639
10	64.89			8.89	17.8		-13.70	- 7.8		- 4.81	- 2.14	
		200	14,000			1,556			-3,352			-1,796
12	75.15			6.67	13.3		-19.82	11.3		-13.15	- 5.85	
		200	15,289			648			-3,342			-2,694
14	77.78			- 0.19[a]	- 0.4		-13.61	- 7.8		-13.80	- 5.97	
		200	15,319			1,500			-3,527			-2,027
16	75.37			15.19	30.4		-21.67	-12.4		- 6.48	- 2.88	
		200	14,563			916			-2,926			-2,010
18	70.22			- 6.02	-12.0		- 7.59	- 4.3		-13.61	- 6.05	
		200	14,207			- 353			- 490			- 843
20	71.89			2.41	4.8		2.69	1.5		5.10	2.27	
		200	15,496			- 259			407			148
22	83.11			- 5.00	-10.0		1.39	0.8		- 3.61	- 1.62	
		200	17,556			-1,611			1,213			- 398
24	92.44			-11.11	-22.2		10.74	6.1		- 0.37	- 0.16	
		200	18,652			- 741			287			- 454
26	94.07			3.71	7.4		- 7.87	- 4.5		- 4.16	- 1.85	
		200	17,289			1,361			241			1,602
28	78.78			9.91	19.8		10.28	5.9		20.19	8.96	
		200	17,022			1,389			768			2,157
30	91.44			3.98	8.0		- 2.59	- 1.5		1.39	0.62	
		200	18,119			- 37			- 815			- 852
32	89.74			- 4.35	- 8.7		- 5.55	- 3.18		- 9.90	- 4.40	
		200	20,867			- 837			-1,472			-2,309
34	118.93			- 3.98	- 8.0		- 9.17	- 5.25		-13.15	- 5.85	
		200	24,207			64			-2,500			-2,436
36	123.11			4.63	9.3		-15.83	- 9.1		-11.20	- 4.97	
			23,141			- 657			- 833			-1,490

38	108.33	200										
40	76.03	200	18,437	-11.20	-22.4	1,685	7.50	4.3	-2,463	- 3.70	- 1.65	- 778
42	97.40	200	17,348	28.06	56.1	2,037	-32.13	-18.4	-2,768	- 4.07	- 1.81	- 731
44	93.96	200	19,141	- 7.69	-15.4	- 778	4.44	2.54	18	- 3.25	- 1.45	- 760
46	108.81	200	20,281	- 0.09	- 0.2	- 601	- 4.26	- 2.44	-1,768	- 4.35	- 1.93	-2,369
48	84.81	200	19,363	- 5.93	-11.9	- 805	- 3.43	- 1.96	-1,000	- 9.36	- 4.17	-1,805
50	80.33	200	16,519	- 2.13	- 4.3	601	- 6.57	- 3.8	- 648	- 8.70	- 3.86	- 47
52	74.44	200	15,481	8.15	16.3	2,120	0.09	0.05	296	8.24	3.66	2,416
54	63.89	200	13,837	13.06	26.1	2,074	2.87	1.64	250	15.93	7.09	2,324
56	77.07	200	14,096	7.68	15.4	- 463	- 0.37	- 0.21	620	7.31	3.25	157
58	68.37	200	14,548	-12.32	-24.6	-2,630	6.57	3.8	4,018	- 5.75	- 2.56	1,388
60	87.11	200	15,548	-13.98	-28.0	- 7	33.61	19.2	3,981	19.63	8.74	3,974
62	113.63	200	20,074	13.91	27.8	974	5.83	3.3	2,204	19.74	8.78	3,178
64	95.52	200	20,919	- 4.17	- 8.3	- 398	16.20	9.3	3,990	12.03	5.35	3,592
66	50.89	200	14,637	0.19	0.4	2,435	23.70	13.6	1,630	23.89	10.62	4,065
68	84.81	200	13,607	24.17	48.3	4,787	- 7.41	- 4.2	-3,120	16.76	7.45	1,667
70	57.26	200	14,207	23.70	47.4	7,361	-23.80	-13.6	-6,889	- 0.10	- 0.04	472
72	142.52	200	19,978	49.91	99.8	5,278	-45.09	-25.8	-10,361	4.82	2.14	-5,083
74	149.41	200	29,193	2.87	5.7	1,407	-58.52	-33.4	-15,000	-55.65	-25.10	-13,593
76	149.81	200	29,926	11.20	22.4	2,815	-91.48	-52.3	-19,082	-80.28	-35.70	-16,267
78	111.04	200	26,089	17.32	34.6	2,648	-99.35	-56.7	-17,759	-82.03	-36.50	-15,111
80	48.89	200	15,993	9.17	18.3	3,037	-78.24	-44.7	-12,435	-69.07	-30.70	-9,398
82	8.22	200	5,711	21.20	42.4	2,981	-46.85	-26.8	-6,518	-25.65	-11.41	-3,537
84	0	200	822	8.61	17.2		-18.33	-10.5		- 9.72	- 4.32	
Total	83.3	8,300	691,326	5.26	10.5	43,682	-12.33	- 7.0	-102,414	- 7.08	- 3.15	-58,732

[a] - denotes erosion. No sign denotes accretion.

in composition consisting largely of gravel. The fill was finer with a median diameter west of Sherwood Point averaging 0.73 mm, varying from 0.36 mm to 1.50 mm and east of the point averaging 0.95 mm, varying from 0.28 mm to 1.80 mm.

Comparison of the slopes before and after fill placement by means of their steepness characteristics is made in Table 18. The comparison shows that the fill was generally placed initially on a flatter slope than the original beach and it has subsequently tended to steepen.

TABLE 18.—COMPARISON OF STEEPNESS CHARACTERISTICS, SHERWOOD ISLAND STATE PARK

Profile	Steepness Characteristics		
	Before Fill June 1957	After Fill July 1957	After Fill June 1958
A+10	.040	.019	.020
B+6	.031	.039	.050
C+2	.042	.021	.049
D+4	.057	.011	.017
E+4	.021	.012	.014
Average	.038	.020	.030

Surveys of beach profiles shown on Fig. 19 were repeated on June 1957 before filling, and on July 1957, November 1957, and June 1958 after filling. Changes in the positions of the mean high water lines are shown on Fig. 20. They consisted of a recession of over 100 ft at the tip of Sherwood Point, accretion generally to the west of up to 40 ft, and small irregular changes to the east with recession up to 50 ft near the east end.

Volumetric changes are shown in Table 19. Heaviest losses of fill occurred in the vicinity of Sherwood Point. Other smaller losses occurred along the east end of the beach east of profile D+4. Gains occurred elsewhere. The net total losses occurred at an average annual rate of over 26,000 cu yd. The annual rate of loss estimated prior to fill placement[10] was 4,600 cu yd.

[10] Area 1—Ash Creek to Saugatuck River, Conn., Beach Erosion Control Study, House Document No. 454, 81st Congress, 2nd Session, 1950.

CONCLUSIONS

Some continuing patterns of behavior are apparent from study of the beach changes. The results of a predominant direction of littoral drift in causing movement of fill are apparent at Hampton Beach, White Sand Beach, Prospect Beach, Woodmont Shore, Gulf Beach, and at Sherwood Island State Park. West Silver Sands Beach and Seaside Park, both concave, pocket shaped beaches exhibit a pattern of movement of material from their ends towards the middle.

Rates of change of beach fills during the comparatively short periods measured, greatly exceeded those for beaches in their natural state determined from their behavior over longer periods. In some cases this was obviously due to the changed conditions imposed by the placement of fine material along an exposed shore which had previously been protected against rapid losses by the coarse material of which it was composed. This was the case along the north end of Hampton Beach, at Gulf Beach, and at Sherwood Island State Park in the vicinity of Sherwood Point. The increase in the quantity of fine material that could be transported by littoral forces over that formerly available is believed to be at least partly responsible for the more rapid changes at Seaside Park. Rates of change at Plum Island, Misquamicut Beach and possibly the area at Hampton Beach below low water are believed to result from large variable movements of material which normally occur in these areas. These rates will probably be reduced if they are averaged with changes which occur over longer periods. The larger rates of loss at Woodmont Shore and to a lesser degree at White Sand Beach resulted partly from assumption by the fills of a flatter, more natural slope consistent with the new conditions created by the fills. The reason for net gains rather than losses at West Silver Sands Beach and Prospect Beach is not apparent. The history of erosion at these beaches leads to the belief that this condition is temporary. Rates of erosion at some of the beaches, which were measured over short periods, may not be indicative of average losses over long periods. The difficulties encountered in accurately predetermining the rates of loss of beach fills are inherent in this problem since the only basis for solution usually available is the behavior of the beach as it existed before improvement, a condition quite often unlike that afterwards. Quantitative information covering past behavior of beach fills covering a wide variety of conditions could be a useful guide for the beach erosion control engineer in estimating future changes on similar projects.

TABLE 19.—VOLUMETRIC CHANGES SHERWOOD ISLAND STATE PARK, CONN.

Profile	Original Fill July 1957			Volumetric Changes July 1957 - Nov 1957			Volumetric Changes Nov 1957 - June 1958			Volumetric Changes July 1957 - July 1958		
	Volume, cu yd per linear ft	Length Shore, ft	Total Volume, cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd	cu yd per linear ft	cu yd per linear ft per yr	Total cu yd
A + 4	0											
		--	9,424									
A + 6	64.44			3.38	10.1		1.60	2.7		4.98	5.4	
		200	14,636			1,268			758			2,026
A + 8	81.96			9.30	27.9		5.99	10.3		15.29	16.7	
		200	17,015			824			320			1,144
A + 10	87.82			- 1.07[b]	- 3.2		- 2.79	- 4.3		- 3.86	- 4.2	
		200	15,194			468			166			634
A + 12	64.12			5.74	17.2		4.44	7.6		10.18	11.1	
		200	17,322			575			462			1,037
A + 14	109.10			0	0		0.18	0.3		0.18	0.2	
		200	20,071			- 65			320			255
A + 16	91.61			- 0.65	- 2.0		3.02	5.2		2.37	2.6	
		--	15,791			802			1,029			1,831
B	97.48			10.25	30.8		9.30	15.9		19.55	21.3	
		--	19,551			1,086			1,381			2,467
B + 2	129.83			2.37	7.1		6.75	11.6		9.12	9.9	
		200	18,619			1,126			865			1,991
B + 4	130.43			8.89	26.7		1.90	3.3		10.79	11.8	
		200	25,434			628			- 581			47
B + 6	123.91			- 2.61	- 7.8		- 7.70	-13.2		-10.31	-11.2	
		200	22,513			-1,037			-2,169			-3,206
C - 1	101.21			- 7.76	-23.3		-13.99	-24.0		-21.75	-23.7	
		--	16,228			-1,847			-1,928			-3,775
C - 2	125.70			-18.07	-54.2		-12.98	-22.2		-31.05	-33.9	
		--	14,387			-2,552			-2,415			-4,967
C - 3	95.63			-24.89	-74.7		-24.18	-41.4		-49.07	-53.5	
		--	14,746			-3,413			-3,806			-7,219
C - 4	131.26			-27.62	-82.9		-34.37	-58.9		-61.99	-67.6	
		200	30,009			-2,963			-6,282			-9,245

C + 2	168.83	200	28,664	- 2.02	- 6.1	190	-28.44	-48.7	-3,478	-30.46	-33.2	-3,288
C + 4	117.81	200	22,033	3.91	11.7	978	- 6.34	-10.9	- 681	- 2.43	- 2.7	297
C + 6	102.52	200	22,293	5.87	17.6	545	- 0.47	- 0.8	- 130	5.40	5.9	415
C + 8	120.41	200	23,194	- 0.42	- 1.3	284	- 0.83	- 1.4	- 47	- 1.25	- 1.4	237
C + 10	111.53	--	18,727	3.26	9.8	384	0.36	0.6	16	3.62	3.9	400
D	96.52	--	17,189	1.02	3.1	380	- 0.18	- 0.3	192	0.84	0.9	572
D + 2	100.0	200	20,667	2.19	6.6	24	- 2.37	4.1	421	4.56	5.0	445
D + 4	106.7	200	20,756	- 1.96	- 5.9	- 95	1.84	3.2	- 356	- 0.12	- 0.1	- 451
D + 6	100.9	200	17,830	1.02	3.1	190	- 5.39	- 9.2	-1,582	- 4.37	- 4.8	-1,392
D + 8	77.4	200	14,816	0.89	2.7	- 41	-10.43	-17.9	-1,695	- 9.54	-10.4	-1,736
D + 10	71.9	200	15,267	- 1.30	- 3.9	- 836	- 6.52	-11.2	- 978	- 7.82	- 8.5	-1,814
D + 12	81.7	--	13,607	- 0.71	- 2.1	-1,168	- 3.26	- 5.6	- 485	- 3.97	- 4.3	-1,653
E	69.4	--	13,548	- 5.93	-17.8	-1,056	- 2.13	- 3.7	- 161	- 8.06	- 8.8	-1,217
E + 2	91.9	200	15,941	- 6.87	-20.6	- 551	0.18	0.3	47	- 6.69	- 7.3	- 504
E + 4	67.6	200	12,807	1.36	4.1	- 569	0.30	0.5	- 521	1.66	1.8	-1,090
E + 6	60.5	200	8,918	- 7.05	-21.2	- 575	- 5.51	- 9.4	- 551	-12.56	-13.7	-1,126
E + 8	28.7			1.30	3.9		0	0		1.30	1.4	
Total	93.6[a]	--	557,197	- 1.21[a]	- 3.64[a]	-7,016	- 3.67[a]	- 6.3[a]	-21,869	- 4.85	- 5.3	-28,885

[a] Based on scalled shore length of 5,950 ft.
[b] Denotes erosion. No sign denotes accretion.

ACKNOWLEDGMENTS

The assistance of John T. Hannon of the Massachusetts Department of Public Works, Henry Ise' of the Rhode Island Department of Public Works, John J. Curry, Merwin E. Hupfer, A.M. ASCE, of the Connecticut Water Resources Commission who supplied information and data, Thomas B. Finneran, who made many of the computations, and Arthur L. Devine, who prepared the figures, is gratefully acknowledged.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

SYNTHETIC MATERIAL FILTERS IN COASTAL PROTECTION

by Hans A. Agerschou,[1] A. M. ASCE

SYNOPSIS

Protective filters for granular soils normally consist of layers of graded sand, gravel, and rock materials which are often expensive.

This paper discusses the possibilities of substituting "synthetic material filters," especially glass fiber and plastic filters, for graded material filters in coastal protective structures, harbor, canal, and river structures.

INTRODUCTION

The term "synthetic material filters" in the title refers to filters made from synthetic fibers such as glass fiber, nylon, plastic, and others. This paper deals specifically with the application of glass fiber and plastic materials.

Experience has demonstrated the hard way that coastal protection, harbor, canal, and river structures, such as revetments, levees, sea walls, groins, and jetties, built on granular soil or with granular soil as backfill are unstable unless the soil is prevented from being washed out.

The best protection is obtained by using a design which is permeable for water but does not allow backfill material to pass through it. Such a protection usually consists of a graded filter of coarser material (usually sand, gravel and or quarry waste material) resting on the fill material. The filter should:

1. be fine enough to prevent fill particles from passing through it, thus causing settling of and damage to the structure; and 2. allow seepage flow to pass

Note.—Discussion open until July 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 1, February, 1961.

[1] Asst. in Research, Coastal Engrg. Lab., Univ. of Florida, Gainesville, Fla.

through it without too much head loss which could build up larger pressure differences with damaging effect to the structure.

In may cases filters also supply weight which prevents a "quick condition" from occurring in a granular soil exposed to an upward flow.

Theoretical and practical filter problems have been investigated earlier by G. E. Bertram,[2] F. ASCE, F. Engelund,[3] E. Lindquist,[4] F. ASCE, M. Muskat,[5] E. Reinius,[6] F. E. Richart, Jr.,[7] F. ASCE, and J. H. Schmertmann,[7] A. M. ASCE, and Karl Terzaghi,[8,9] Hon. M. ASCE.

The "classic" filter design criterion for graded material filters, which is found subsequently under the heading "Requirements for a Filter," was suggested by Mr. Terzaghi and recommended by the United States Waterways Experiment Station.[10]

An investigation of gravel blankets as protection against wave action has been carried out by Enos J. Carlsson.[11] D. W. Taylor[12] has discussed filter problems.

The previously-mentioned graded material filters are, unfortunately, expensive, particularly where suitable filter material is not available locally.

In 1957 a Dutch engineer, Jan Leendertse, employed by the Coastal Engineering Laboratory of the University of Florida, suggested that glass fiber insulation mats be used on an experimental basis as filter, substituting for graded material filter.

After the durability in sea water had been tested by a manufacturer in November, 1957 (see "Durability of Glass Fiber Mat in Sea Water" shown subsequently) the first application of glass fiber mat as a filter was made at Palm Beach, Florida, in May, 1958, in a sloping interlocking block revetment described under the heading "Glass Fiber Mat Filter."

Since then the glass fiber mat filter has been used in similar revetments in Florida, and all the applications have so far been successful. Glass fiber cloth and plastic cloth have been used under rubble mounds and concrete bag revetments some places in Florida since the Fall of 1958. A certain plastic cloth (described under the heading "Plastic Cloth") has so far proved feasible.

Some experiments with glass fiber mat, welded glass fiber cloth, and woven plastic cloth were conducted at the Coastal Engineering Laboratory

---

2 "An Experimental Investigation of Protective Filters," by G. E. Bertram, Harvard Graduate School of Engrg. Pub. No. 267, Cambridge, Mass., 1940.

3 "On the Laminar and Turbulent Flows of Ground Water through Homogeneous Sand," by F. Engelund, Transactions, Danish Acad. Tech. Sciences, No. 3, Copenhagen, Denmark, 1953.

4 "On the Flow of Water through Porous Soil," by E. Lindquist, Proceedings, First Congress of Large Dams, Stockholm, Sweden, 1933.

5 "The Flow of Homogeneous Fluids through Porous Media," by M. Muskat, New York, McGraw-Hill Book Co., Inc., 1932.

6 "The Stability of the Upstream Slope of Earth Dams," by E. Reinus, Swedish State Committee for Building Research Bulletin No. 12, 1948.

7 "The Effect of Seepage on the Stability of Sea Walls," by F. E. Richart, Jr. and J. H. Schmertmann, Proceedings, Sixth Conf. of Coastal Engrg., Gainesville, Fla., 1957.

8 "Theoretical Soil Mechanics," by K. Terzaghi, Wiley, New York, 1943.

9 "Soil Mechanics in Engineering Practice," by K. Terzaghi and R. B. Peck, Wiley, New York, 1948.

10 "Investigation of Filter Requirements for Underdrains," by U. S. Waterways Experiment Station, (revised) Tech. Memo. No. 183-1, 1941.

11 "Gravel Blanket to Prevent Wave Erosion," by Enos J. Carlsson, Proceedings, ASCE, Vol. 85, No. HY 5, May, 1959.

12 "Fundamentals of Soil Mechanics," by D. W. Taylor, Wiley, New York, 1948.

of the University of Florida, as described under the heading "Wave Tank Experiments with Synthetic Filters."

Experiments with nylon sheets as bottom protection during closing of tidal gaps in dikes have been carried on in Holland since 1956. The experiments are described in various Dutch publications.

REQUIREMENTS FOR A FILTER

A filter material must meet the following requirements:

1. Impermeability for soil—it must be fine enough (the pores small enough) to prevent particles in the layers below from being washed out.
2. Permeability for water—it must be coarse enough (the pores in it must be large enough) to make sure that the head loss for the seepage flow through it and, therefore, the seepage forces are sufficiently small.
3. Strength—a certain initial strength is necessary so that the filter can be handled and heavy material placed on it in an economical way without damaging it. Furthermore, the material must be able to withstand pressure and abrasion caused by wave action or other forces.
4. Durability—the filter material should not change its permeability characteristics with time and must be insoluble.

This means that the filter has to withstand:

a. dumping of rock or blocks as part of a rubble mound construction;
b. shock pressures from wave action;
c. movements of rock or blocks caused by wave action;
d. sea water and or fresh water;
e. chemicals and biological growths (molds, algae, and so forth) in sea water, fresh water, or in the soil under the filter;
f. varying temperatures including frost temperatures;
g. continued change between wet and dry conditions; and
h. storage exposed to air and humidity.

For the preceding requirements (1 and 2), a criterion, suggested by Terzaghi and recommended by the United States Waterways Experiment Station,[10] for graded material filters may be written:

$$\frac{D\ 15\ \text{(of filter)}}{D\ 85\ \text{(of foundation)}} < (4\text{-}5) < \frac{D\ 15\ \text{(of filter)}}{D\ 15\ \text{(of foundation)}}$$

This means that the diameter which corresponds to 15% on the grain-size curve for the filter should be:

(a) smaller than four to five times the diameter corresponding to 85% on the grain-size curve for the material which is supposed to be protected; and
(b) larger than four to five times the diameter corresponding to 15% on the grain-size curve for the material which is supposed to be protected.

G. E. Bertram[2] has proved by laboratory investigations that the grain size of a uniform filter material may be as much as ten times that of a uniform foundation material before there is appreciable leakage of material.

The preceding specifications are applicable only to filters consisting of numerous layers of grains, that is, filters of certain thickness.

The openings in a thin filter as a general rule ought to be smaller than the majority of sand grains in the sand material which is supposed to be

protected. The grain-size diameters a thin filter with a certain size of openings is able to protect can be investigated by washout experiments as described subsequently under the heading "Washout Experiments."

DESCRIPTION OF GLASS FIBER MAT GLASS FIBER CLOTH, AND PLASTIC CLOTH

Glass Fiber Mat.—The glass fiber mat in question consists of a 1 in. to 2 in. thick porous layer of glass fibers (fused oxides) bonded together with a

FIG. 1.—GLASS FIBER FILTER MAT

phenolic resin faced with a thin glass fiber layer which is bonded together with a suitable binder (for example, neoprene).

The thin layer gives the mat some tensile strength making it possible to handle the mat without making holes in it or tearing it. This glass fiber mat material is on the market in 4 ft wide rolls (see Fig. 1).

Glass Fiber Cloth.—Welded glass fiber cloth is woven from glass fiber continuous filament yarns which are coated with a plastic. After the cloth is woven it is passed through an oven to soften the resin on each yarn so that at each point where yarns cross they will adhere to each other.

A welded glass fiber cloth with a suitable fine mesh is under development.

Plastic Cloth.—A type of plastic cloth which has been used in Florida since 1958 consists of thin and rather wide vinylidene chloride monofilament yarns woven closely together without use of a binder or welding. The edges of the cloth are bent around and seamed to avoid separation. Such a non-welded cloth can only be used when it is thus carefully seamed along all edges. Holes which develop in such material are, because of the fraying and separating possibility, more detrimental than holes in welded glass fiber cloth. This cloth is on the market in 6 ft wide rolls.

EXISTING STRUCTURES WITH SYNTHETIC FILTERS

Glass Fiber Mat Filter.—The first glass fiber mat filter was used in an interlocking block revetment on the ocean in Palm Beach, Florida, in May, 1958.

A cross section and a top view of this construction is shown in Fig. 2, where it can be seen that the interlocking blocks, as well as the burlap bags in the apron, are resting on glass fiber mats.

The apron (the bottom) of the revetment was constructed in a trough averaging 10 ft in width which was covered with mats upon which were laid three layers of burlap bags with ready-mix concrete. The bags were secured with glass fiber rods 5/8 in. in diameter, and 20 in. long, driven down through the bags before the dry-mix concrete hardened.

Fig. 3 shows the apron of the revetment under construction. Mats 4 ft wide by 40 ft long were rolled out lengthwise along the revetment with an overlapping of about 10 in. and the ready-mix concrete burlap bags were placed on top of the these mats. In the background is an "adjustable" groin, consisting of prestressed concrete H-king piles and removable Florida yellow pine boards. (This groin does not extend beyond the mean low-water line). Fig. 4 shows the finished revetment and part of an adjustable groin.

Since its completion in May, 1958, the revetment has been exposed to northeasterly storms in the fall and winter of 1958 and to heavy swell action in September, 1959, when hurricane "Gracie" passed by in the Atlantic. So far this light and economical construction has worked satisfactorily. Meanwhile, continued erosion of the dune slope on both sides presents a problem of danger of flanking which has to be taken care of in the design and before slope erosion has developed too far. Concrete bags have so far been used for such protection. Concrete sheet piling may be necessary later.

The same revetment type with mat filter has been used with equal success at Delray Beach (south of Palm Beach) on the lower Florida East Coast.

Glass Fiber Cloth.—No applications of welded glass fiber cloth filters have been made so far, but it is considered useful if the openings are reduced considerably from the present 24 by 24 mesh. The size of the openings depends, of course, on the grain sizes which are supposed to be protected. Nonwedled glass fiber material or material with a loose weave separates when subjected to salt water or wave action and is, therefore, not useful as filter.

Plastic Cloth.—Plastic cloth filters (described previously) have been used under rubble mounds and concrete bag revetments in several places on the lower Florida East and West Coasts and elsewhere within the last year. Cloth which was uncovered recently did not show any signs of changed characteristics after 12 months.

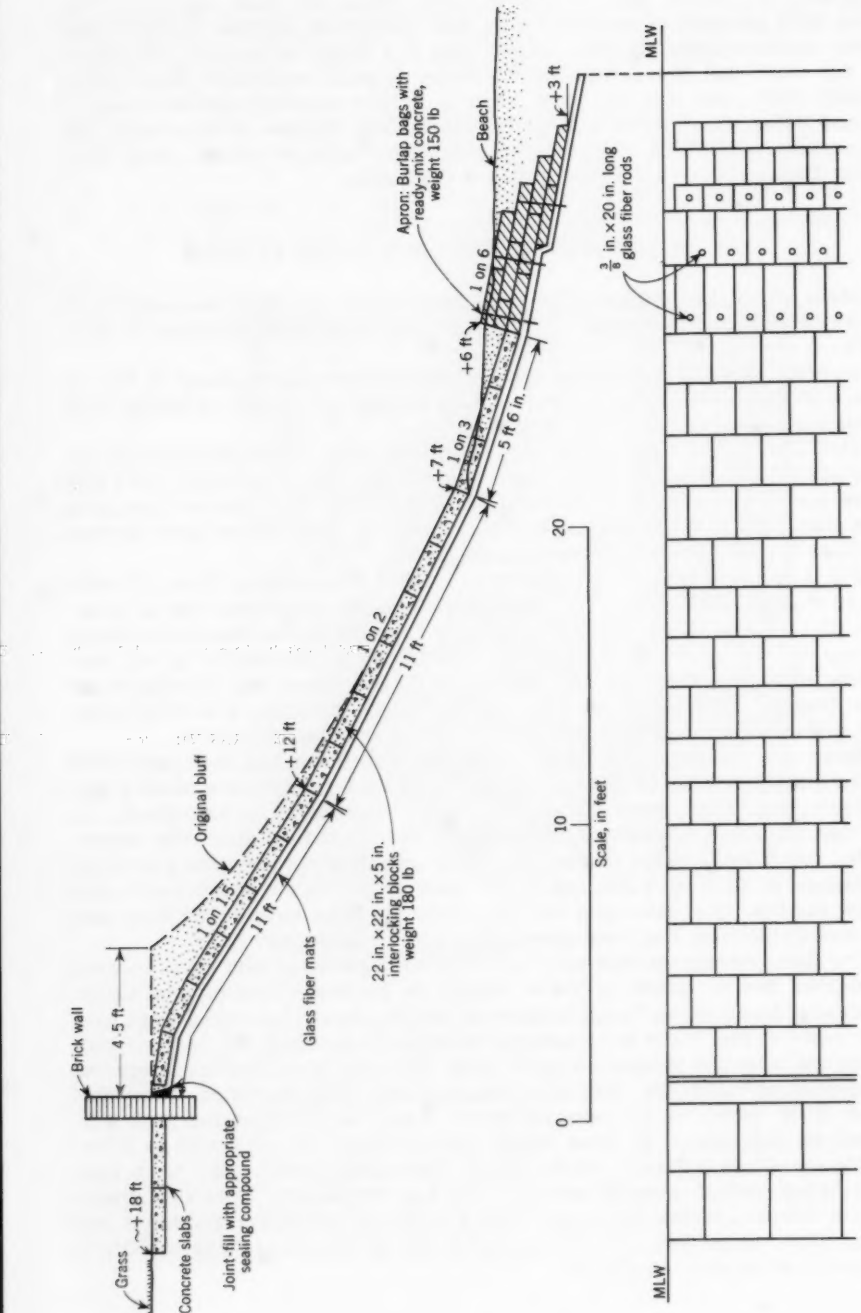


FIG. 2.—INTERLOCKING BLOCK REVETMENT

FIG. 3.—APRON OF REVETMENT UNDER CONSTRUCTION

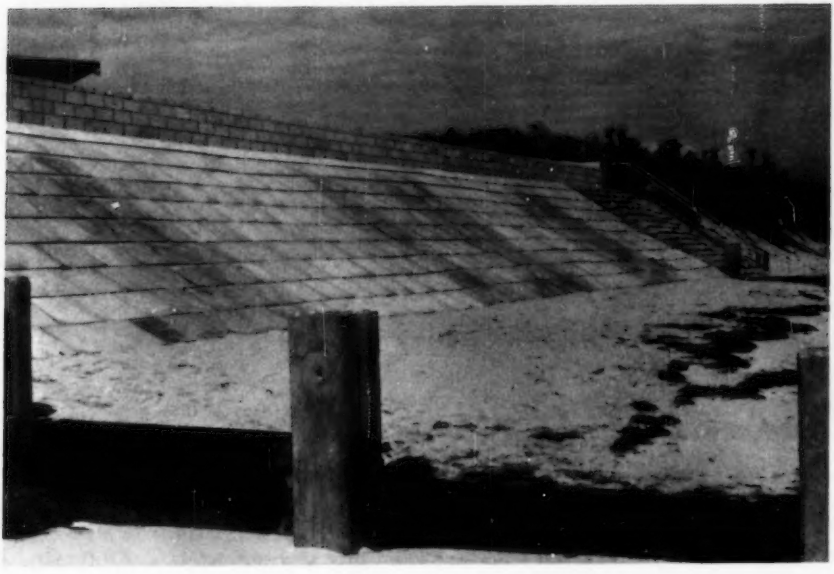


FIG. 4.—FINISHED REVETMENT AND ADJUSTABLE GROIN

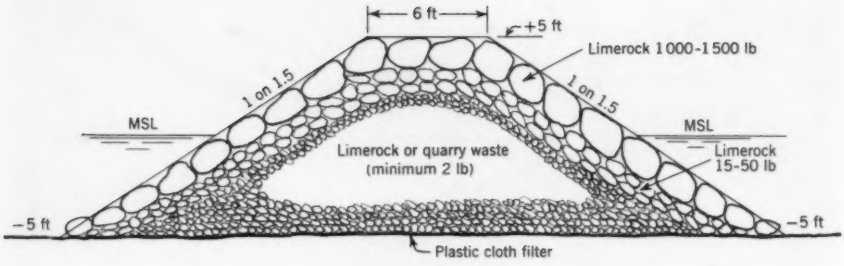


FIG. 5.—RUBBLE MOUND JETTY

FIG. 6.—MODELS OF INTERLOCKING BLOCK REVETMENTS AFTER HAVING BEEN EXPOSED TO WAVE ACTION

Fig. 5 shows plastic cloth used as bottom protection filter in a jetty which is under construction on the lower Florida West Coast.

WAVE TANK EXPERIMENTS WITH SYNTHETIC FILTERS

In the spring of 1959 some experiments of qualitative nature with synthetic filters were performed in the 6 ft wide, 4 ft deep wave tank at the Coastal Engineering Laboratory of the University of Florida.

The experiments were partly wave action tests on models of a few types of revetments, sea walls, and jetties, with and without synthetic protective filters and partly washout tests for glass fiber and plastic cloth under almost "prototype conditions."

Wave Action Tests on Models of Revetments, Sea Walls, and Jetties.—All designs tested assume that the actual construction is built on sand material. Most beaches in the United States are built up of 0.2 mm to 0.4 mm sand. In Florida, however, the lower Gulf beaches have smaller sand particles ranging in size from 0.1 to 0.2 mm.

The direction of wave propagation was perpendicular to all structures tested. The models were built with a fixed bottom in front of them, thereby preventing a beach being built up on the structures at the beginning of each experiment. Such development would create unrealistic conditions for the structures tested.

The forces acting in the field (prototype) are:

1. gravity forces which are always present;
2. inertia forces because of moving water; and
3. forces caused by the interaction between flow and (sand) particles during flow or seepage through the material.

The same forces ought to be acting in the model and the power ratio between them ought to be the same.

If Froude's model law, which is valid when gravity and inertia only are in action, is applied using a certain length scale $1 : \lambda$, the velocity and time scales are $1 : \sqrt{\lambda}$.

Froude's law makes pressure heads follow the length scale. Accordingly head loss per unit length is the same for prototype and model.

For flow of water through porous media such as sand the following law is valid:

$$I = \frac{\alpha \nu}{g d^2} V + \frac{\beta}{g d} V^2 \quad \ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which I is the hydraulic gradient (head loss per unit of length; V is the velocity; ν is the viscosity; g is the acceleration of gravity; d is the grain size diameter; and α and β the dimensionless factors, the values of which depend on the porosity, the grain shape, and so forth.

Where very low water velocities occur (only seepage) the first term of Eq. 1 is predominant and makes the grain size scale $\sim 1 : \sqrt[4]{\lambda}$. Accordingly Froude's law is invalid for the grain-size scale (viscosity phenomena are involved). On a slope covered with blocks or rock (no filter or a thin filter), the sand closest to the surface is exposed to higher flow velocities caused by wave uprush. This makes the second term predominant and the grain-size

scale $\sim 1:\lambda$ in agreement with Froude's law. This is probably not the case when a thick mat filter is used. The velocity of the uprush flow will be reduced considerably in the mat and higher velocities might not reach the underlying sand.

The grain-size scale in a model, where no filter or a thin filter is used, should therefore vary from $1: \sqrt[4]{\lambda}$ at some distance below the surface layer to $1:\lambda$ in the surface layer which is exposed to wave uprush. If a thick mat filter is used the grain-size scale probably ought to be $1: \sqrt[4]{\lambda}$.

In the model studies the actual prototype glass fiber and plastic filter materials were used, together with "model sand" (average diameter $d_{50\%}$ = 0.08 mm) in some experiments thus increasing the possibility for washout and thereby making the experiments unfavorable for the filter material tested. In other experiments prototype sand (average diameter $d_{50\%}$ = 0.20 mm) was used.

The results of the wave action tests can be summarized as follows: as expected, sand was immediately washed out from structures which did not have a filter or which had a 24 mesh glass fiber filter, resulting in cavities in and large settlements to the structures which in some cases collapsed. Structures with glass fiber mat or plastic cloth (as described previously) filters were not damaged by the same wave action. Fig. 6 shows two interlocking block revetments after having been exposed to wave action. The left side revetment has no protective filter. The right side revetment has a glass fiber mat filter under the blocks. Otherwise, the two revetments are alike with a toe protection consisting of a sheet piling and a rubble mound. On top of the revetments are vertical wave screens.

The revetment without filter collapsed as described previously. The revetment with the filter was not damaged.

Washout Experiments.—Besides the experiments with models of structures, some special "washout" experiments were performed.

A box consisting of three sand-filled compartments, 22.5 in. long, 6.5 in. wide and 1 in. deep, which had five sand-tight walls and the upper side covered with welded glass fiber, 24 by 24 mesh cloth or plastic cloth (described previously) was laid on a 1:2 slope and exposed to wave uprush with a maximum velocity of 5 ft per sec which corresponds fairly well to prototype conditions under moderate wave action.

Glass fiber 24 by 24 mesh was tested with two kinds of sand, average diameters $(d_{50\%})$ 0.08 mm and 0.22 mm. The third compartment was filled with sand from the lower Atlanti Coast of Florida having an average diameter of 0.4 mm. After having been exposed to wave uprush for eight minutes all the fine sand, 65% of the medium sand, and 25% of the coarse sand was washed out.

Plastic cloth was tested with the same fine sand $(d_{50\%} = 0.08$ mm) and the same medium sand $(d_{50\%} = 0.22$ mm) in two compartments. After 2 hr uprush action, no sign of washout was visible and the experiment was stopped. The sane samples were dried, and weighing confirmed that no washout had occurred.

STRENGTH AND DURABILITY TESTS ON GLASS FIBER MAT, GLASS FIBER CLOTH, AND PLASTIC CLOTH

Rock Dumping Tests.—Dry sheets of welded glass fiber 24 by 24 mesh and 14 by 18 mesh cloth and plastic cloth were spread out on a leveled sand area

and a 200 lb piece of rock was fastened to the hook on a mobile crane and dumped repeatedly on the sheets from varying heights up to 15 ft. The results were that plastic cloth was able to resist this treatment better than glass fiber 24 by 24 mesh, which in turn was considerably better than glass fiber 14 by 18 mesh.

Tensile Strength and Wear Strength for Dry Cloth Materials.—The tensile strength and wear tests were performed by manufacturers' laboratories which determined the tensile strength for dry glass fiber 24 by 24 mesh cloth to about 180 lb per in. for both warp and filling. For dry plastic cloth tensile strengths of about 150 lb per in. for warp and about 80 lb per in. for filling have been measured.

The tensile strength mentioned is the ultimate strength at the point of failure.

Wear tests were performed in a special wear machine with rotating "wear cylinders."

Glass fiber 20 by 20 mesh cloth was worn out after 453 cycles and plastic cloth after 614 cycles. A finer mesh glass fiber cloth will probably have the same weat strength as plastic cloth.

Durability of Glass Fiber Mat in Sea Water.—The durability of mat material in sea water was tested by a manufacturer. The summary of the test report dated November, 1957, stated that after exposing the samples to accelerated test conditions designed to simulate the effect of twenty years exposure to sea water, the samples appeared to retain sufficient mechanical strength and integrity to indicate that the application was worth investigating on an applied basis. Because it was impossible to duplicate all conditions that would be encountered in use, these observations were intended only as a guide to further investigations.

EXAMINATION

Table 1 is an attempt at grading the three synthetic filter types which have been described here with respect to essential requirements mentioned under the heading "Requirements for a Filter."

Permeability for water flow depends on the size of the openings in the filter compared with the grain sizes of the granular material. With cohesive, usually very fine, material such as clay and with organic material other factors will be of importance for the permeability.

Synthetic filters cannot be used as protection for grains which will stick to each other or to the filter in such a way that permeability for water decreases until the filter finally is clogged up entirely. In such cases it is necessary to build special drains into the design discharging through holes in the protective layers.

Mat filters have so far proved to be an efficient protection in interlocking block revetments, where the blocks are placed carefully resting with their whole surface on the mat. As mentioned previously the mat material cannot be used under rubble mound constructions. Mat material is not recommended for underwater application due to its buoyancy. It is believed that glass fiber mat is applicable to revetments with expansion and other kinds of joints whether they are concrete or asphalt slopes, concrete walls, or concrete or steel sheet pilings.

The initial strengths for glass fiber cloth and plastic cloth as tested by manufacturers seem to be adequate and further experience is being gained under prototype conditions.

Chemical experts believe that the glass fiber and plastic materials described herein are unaffected by temperatures occurring in nature, sea water, and chemicals or biological growth encountered in sea water, fresh water, or in soil. Inspection of plastic cloth in a concrete bag revetment exposed directly to wave action (lower Florida Gulf Coast) revealed no visible change in the material, not even discoloration, after one year.

TABLE 1.—GRADING THE THREE SYNTHETIC FILTER TYPES

Requirement (1)	Glass Fiber Mat (2)	Glass Fiber Cloth Welded 24 by 24 Mesh (3)	Plastic Cloth (4)
Impermeability for soil	Good even for grains with an average diameter as low as 0.08 mm	Unsatisfactory for grains with an average diameter as high as 0.40 mm	Good even for grains with an average diameter as low as 0.08 mm
Initial strength	Not sufficient for dumping of rock or blocks. Sufficient for interlocking block revetments and similar constructions	Good tensile strength. Fair resistance to dumping of rock	Good tensile strength for warp. Tensile strength for filling could be improved. Tough, good resistance to dumping of rock
Lasting strength	Experience from May, 1958, is so far satisfactory for interlocking block revetment	No experience	Twelve months experience from two rubble mound revetments. No signs of deterioration
Constant permeability characteristics	No changes observed after 18 months	No experience	No changes observed after 12 months

Plastic cloth is a woven nonwelded material. Holes which develop in it may increase in size rather rapidly (there is nothing which will stop them).

CONCLUSIONS

Provided the durability is satisfactory which seems probable judging from the experience which has so far been gained, it is concluded:

1. Glass fiber mat seems to offer an efficient filter protection in all structures where it will not be punctured or torn during construction or later when the structure is exposed to wave action. The application is recommendable to interlocking block revetments, concrete and asphalt revetments, concrete walls, and concrete and steel sheet pilings. Due to its buoyancy it can hardly be used under water. A surface layer with higher tensile strength than found at present is desirable.

2. Plastic cloth seems to offer an efficient protection for almost any kind of granular material provided it is used in sheets which are seamed along all edges. Equal tensile strength for warp and filling is desirable.

3. In structures where the use of a usual graded material filter consisting of sand, gravel, quarry waste, and stones is prohibited by its high cost, a synthetic filter is the best substitute known to the writers.

4. Moreover, synthetic filters may also be useful in dams, levees, for canal and river slope and bottom protection, for making jetties and groins sand tight, and perhaps as storm protection for artificially nourished beaches when well anchored in same at a certain distance below the surface. In emergency constructions, synthetic filters probably have definite advantages because they have a low weight and are easy to handle.

5. In structures such as interlocking block, concrete slab, and asphalt revetments the stability is entirely dependent on avoiding washout cavities which will cause sinking, opening of joints, or breaking of the surface elements and resulting collapse of the structure.

Therefore, high quality workmanship is indispensable for such structures. All joints have to be covered completely with synthetic filter or sealed. Holes and cracks in the surface elements cannot be allowed. These requirements are, of course, as important for flanking protection as for the revetment proper. A structure is never stronger than its weakest member.

ACKNOWLEDGMENTS

The writers wishes to express his appreciation to Per Bruun, Head of the Coastal Engineering Laboratory at the University of Florida, for discussions on the manuscript and many helpful suggestions, and to Rachel Albertson who edited the manuscript.

The experiments with synthetic filters which are described here were sponsored by the Owens-Corning Fiberglas Corporation. Mel R. Greiser, Palm Beach, Florida, furnished part of the photo material for this paper.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

DESIGN OF FOX POINT HURRICANE BARRIER

By Roy S. Martin[1]

SYNOPSIS

This paper describes the Fox Point hurrican barrier the major design problems including the establishing of hurrican tide levels and wave forces, the provisions for passage of estuary flow under normal conditions and at flood periods, the protection of a steam power plant from damaging effects during construction and subsequent thereto, and the provisions necessitated by poor foundation conditions.

INTRODUCTION

Hurricanes have been recorded along the Atlantic coastline since the days of the first Spanish explorers. They have varied in frequency, intensity and course beyond the limits of any recognizable pattern. In the New England area, more than forty storms of a hurrican nature have been recorded since 1900, of which three have been disastrous.

The federal government had established no definite policy of protection against hurricane storms prior to 1954. In 1954 and 1955 there were three particularly damaging storms, of which two moved across New England. Subsequently the Corps of Engineers, U.S. Army in cooperation with other federal agencies was directed to study the behavior and frequency of hurricanes and to consider possible means of preventing loss of lives and property.

The New England Division, Corps of Engineers completed an interim survey report on protection for the Narragansett Bay area in 1957 (Fig. 1).

Note.—Discussion open until July 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 1, February, 1961.

[1] Project Engr., U. S. Army Engr. Div., New England, Corps of Engrs., Waltham, Mass.

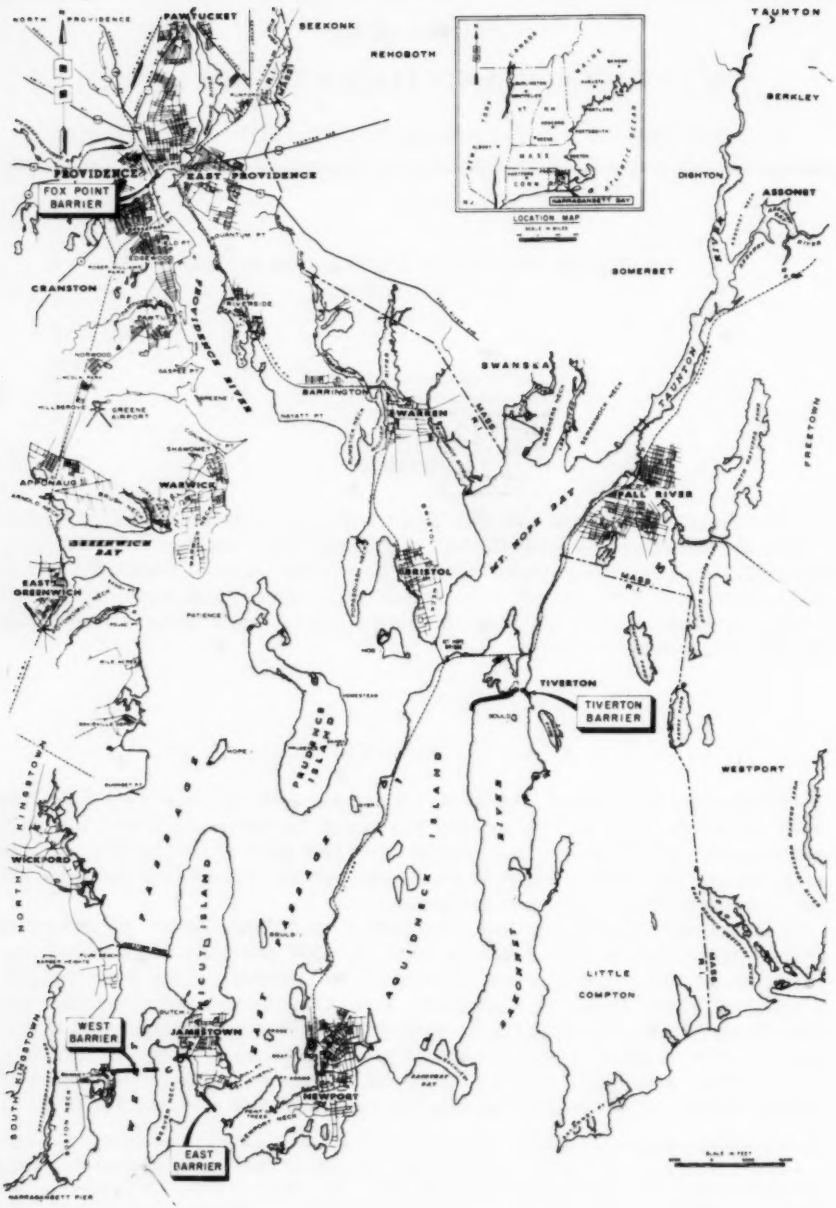


FIG. 1.—GENERAL PLAN OF NARRAGANSETT BAY

This report recommended a hurrican barrier across the Providence River at Fox Point in Providence, R. I., for the protection of the downstream commercial and industrial section.

The Congress of the United States authorized the project in 1958 and took the unusual step of appropriating funds for design the same year.

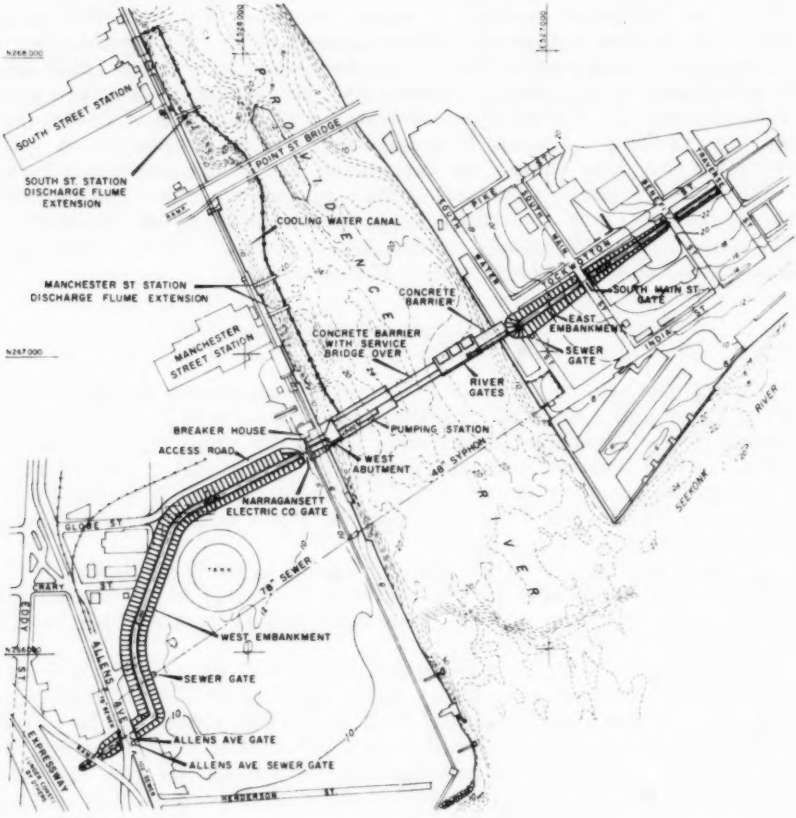


FIG. 2.—FOX POINT HURRICANE BARRIER

Design was completed in December 1960. Construction was initiated in December 1960 and will be effectively completed prior to the 1963 hurricane season.

REPORT STUDIES

In the preparation of survey reports, the object of planning is to determine the most feasible plan of improvement and whether such improvement is economically justified as a federal undertaking.

is economically justified as a federal undertaking. Sufficient investigations are made to develop, in general outlines, a feasible engineering plan and a reliable estimate of costs and benefits.

The survey report plan for hurrican protection for the Narragansett Bay includes two barriers, the Lower Bay barrier closing the three passages to the Atlantic Ocean and the Fox Point barrier at the head of the bay. The Lower Bay Barrier would provide overall protection for the bay, but the effects on navigation, quality of water, fisheries, and recreation require extensive studies before a definite project could be recommended, authorized, and constructed. The Fox Point project will afford a high degree of protection to the valuable commercial and industrial downtown section of Providence, R. I. until these problems are resolved. It will subsequently provide protection against the moderate hurricane tides which will be generated by wind action on the waters behind the Lower Bay barrier.

The site of the Fox Point hurrican barrier as selected in the interim survey report is at the confluence of the Providence and Seekonk Rivers. The alignment is diagonally across the Providence River, continuing on land on each abutment to high ground (Fig. 2).

DESCRIPTION OF PROJECT

The project will include 670 ft of concrete structures across the Providence River and 1,850 ft of rock-armored earth dikes in the land areas connecting to high ground on either side. The design grade of top of the structures is el 25.0 above mean sea level (msl). The concrete river structures will include a pumping station of 7,000 cfs capacity at 20 ft head and a river gate section with three 40-ft wide openings. Three vehicular gate structures will be provided in the dike areas.

A canal will extend from gated openings in the pumping station to the cooling water intakes of the Narragansett Electric Company steam power plants upstream of the barrier. This canal will be formed by partitioning a section of the Providence River.

Earth Dikes.—Rock armored earth dikes will be provided because of their ability to absorb wave forces and to adjust to settlement of the foundation. The top width will generally be 20 ft and side slopes will be 1.5-on-1.

The compacted earth core will be of impervious materials of which there are several sources of borrow within a few miles of the site. The section will include a cutoff trench of similar material to permanent ground water level, a depth of approximately 10 ft. A wood sheet pile cutoff below the trench will extend to satisfactory materials. The cutoff will be of untreated 6 in.-by-10 in. timbers, grooved and splined, extending through the existing fill materials.

The downstream rock protection against wave action has been designed according to established criteria. The surface course will be 7 ft of 2 ton to 3 ton stone. Underlying the stone will be successive layers of bedding stone, filter stone, and filter gravel.

The design storm will produce occasional waves much in excess of the significant wave height of 5.1 ft and the force of the waves will drive water to a height greater than the top of the dike. This condition will occur over a maximum period of 2 1/2 hr at the design still water level. To assure complete protection against over-topping would be uneconomical considering

the character of the protected area behind the dikes and the nearness of the river to which overflow would drain. Rock protection consisting of a 2-ft layer of 75 lb to 150 lb stone on gravel bedding is provided over the top and back slope of the dikes for a maximum 0.5 cfs per ft of overtopping. A nominal drainage ditch is provided at the upstream dike toe.

River Excavation and Backfill.—A layer of weak organic silt, about 20 ft thick, will be excavated from the river and replaced with granular materials.

Pumping Station.—The pumping station will be a reinforced concrete substructure with brick superstructure 213 ft long and 91 ft wide. It will house five 120-in. vertical propellor type pumps and will have an access bay at the west end. The cooling water intake gates will be in the access bay.

The selection of a location near the west bank of the river was governed largely by the economy of locating the colling-water gates in the access bay.

Vertical steel discharge casings will be flared at the overflow end, el 17.5. Use of this type of discharge tube permits a simple and economical structural layout for the substructure and obviates the expense of the fabricated steel, curved siphon discharge which was considered as an alternate. The increase in power requirements over a pump with siphon discharge is not serious in this case because operation will be infrequent. Backflow at extreme hurricane tide levels will be prevented by a hydraulically operated "lid" over the discharge tube.

The impeller and part of the shaft and casing are in the tidal range and will be susceptible to maximum corrosion action of the highly polluted river water. The pump shaft will be inclosed in an oil-filled cylinder and special alloys will be used for impeller and casing.

Electric motors for the pumps, controls, transformers, and other accessory equipment will be located on the operating floor, el 26.0. Controls will be in an enclosed heated area. The remainder of the operating room area will be heated to a lower temperature with gas-fired heaters to minimize condensation.

The access bay is essentially an open well, with the floor at ground level for lifting equipment to the operating floor level. Two 10 ft-by-15 ft vertical lift sluice gates will be installed outside the limits of the well to control flow through intake passages which are below the ground floor.

A centrifugal type unwatering pump with headers to each pump intake chamber will be installed in a dry sump in the east end wall.

Power supply will be obtained at 11 kv from the Narragansett Electric Company substation nearby. The substation is fed from the power station and is interconnected to the New England Electric System. Two separate underground circuits will connect to the pumping station transformers, the second circuit being installed to increase reliability of service.

River Gates.—The river gate structures will include three 40-ft-wide tainter gates, hoist operated from an operating platform at el 35.0, well above maximum wave runup. Sill will be at el - 15.0. Gates are designed to open to a maximum clearance of el $\pm$ 25.0 to permit passage of small boats. The gates will be secured above the corrosive river waters. The trunnions will be above normal high tide elevation. The electrically operated hoists will be remotely controlled from the pumping station. Access will be by a 10-ft bridge to the pumping station.

Consideration was given to various alternate types of closures, including flap gates hinged at the top to swing down for closure, gates normally resting

flat on the bottom of the gate structure to be raised for closure and vertical lift gates. Tainter gates were selected for economy of construction, economy and convenience of maintenance, ease of operation, advantageous vertical clearance, and for relative freedom from corrosion.

Barrier Concrete Sections.—Concrete walls of a gravity section will be provided to complete the closure across the river. The walls will be keyed into the existing river abutments and the earth dikes will form a wrap-around The abutment section adjacent to the west end of the pumping station will be a cellular structure because of the unusual width necessary for access to the pumping station.

Vehicular Gates.—Three vehicular gates are provided, two for passage of traffic on city streets and the third for access between the generating plants and the unloading and coal storage facilities of the electric utility company. The gates will consist of concrete substructures and abutments with steel gates. Due to the heavy traffic, the gates must be kept open as long as safety of the protected area will permit in the imminence of a hurricane tide. Rapid and secure closure is therefore necessary. The gates will be of a swinging type supported at the abutments. An upper hinge and a lower socket pivot will support the gate load. High-strength steel is used to reduce the weight. For the street closures, intermediate A-frame supports are attached to the gates. They will normally be secured flat against the gates but will be opened out and bolted to seats recessed into the pavement when the gates are closed.

Piling.—All structures will be supported on steel bearing piling.

Utilities.—Utilities passing through the barrier include sewer, water, gas and electric lines in the streets and water, steam, air and oil lines in the electric utility company area. Lines of the electric utility company will be relocated through the vehicular gate structure. Utilities in the street will generally remain on existing alignments.

Sewers passing through the barrier are the collectors for the entire city upstream of the site, carrying sewage to the Fields Point pumping station and treatment plant. They are at a relatively low elevation, with invert at - 11 mean sea level at Allens Avenue gate. They are brick or concrete structures, about 75 yr old (as of 1960), and are supported by sheeting or piling. The portions under the barrier will be replaced with reinforced concrete sections, supported on timber piling. Because the sewage plant and collectors will be flooded in case of hurricane tide, the sewers will be gated to prevent backflow.

The flow in the sewers upstream of the barrier will be backed up when the gates are closed. Relief sewers, gated for closure under normal operating conditions, will bypass the backup to the Providence River pool behind the barrier. The bypasses will be at pool elevation, which is higher than normal sewer flow gradients and which will result in interruption of service to down town buildings for the period of flooding which is estimated to be 11 hr maximum. Practically all downtown buildings have backwater gates on their sewers as past storms or interruptions in pumping at Fields Point have been experienced, therefore only temporary invonvenience will result. Bypass of untreated sewage to the Providence River for the flood period is acceptable to local authorities because the plant at Fields Point is subject to hurrican flooding and would not be functioning during the period.

Cooling Water Canal.—The canal will be formed by constructing a panel wall paralleling the river bank to form a channel averaging 65 ft in width.

It will be 1,550 ft long, extending to the upper intakes at the South Street plant. It will constitute a trapezoidal section, conforming to the general existing river cross section. The cross-sectional area is designed to minimize head loss to the intakes and to maintain a non-eroding bottom velocity.

The power plant intake pumps provide constant discharge at variable head. Increased head required additional power for operation. The anticipated head loss of approximately 0.5 ft in the canal will not increase power requirements appreciably.

As the river bottom is somewhat irregular over the length of the canal, some excavation will be necessary to obtain the design cross section. In addition the loose top layer of bed material, which contains a proportionately high proportion of organic silt and erodes readily, will be removed so far as practicable to reduce the quantity of sulphide bearing material that would pass through the power plant condenser tubes at initiation of use of the canal. The depth of the excavation is limited by the fact that the base of the existing rubble masonry river wall is only slightly below water level and essentially rests on the relatively consolidated underlying organic silt layer.

The safe eroding velocity of the underlying, relatively consolidated, organic silt stratum is about 0.5 fps. The maximum design velocity in the canal is slightly under 1.0 fps. It is not practical to enlarge the cross-sectional area sufficiently to assure noneroding velocities for the silt, therefore a 2-ft layer of sand and gravel will be placed 'in the wet' to form a non-eroding lining. It is anticipated that some loose silt remaining after the preventive excavation will be displaced by the dumping of the sand and subsequently eroded, but there is no practical means of avoiding this situation.

The canal wall will be a timber panel structure, supported by vertical H-piles driven to glacial till. The depth of water, the relative narrowness of the Providence River, and the poor foundation conditions make an earth dike divider impractical. The large volume of cooling water combined with the poor foundation conditions made a conduit a costly alternate. The H-piling will be supported by diagonal brace piles and logitudinal walers. Impressed current cathodic protection for the steel is necessary because of the corrosive elements in the water. Steel above low water line will be coated with a protective paint. The canal wall is designed for 2-ft head differential, which is adequate for normal conditions. The selected timber panel wall is more economical in first cost than a steel sheet pile wall and is, as a whole, as durable as steel in water of this character. Marine borers are not present, therefore no protection against their activities is required. Replacement or repair of timber panels or steel piling can be readily accomplished by removing deteriorated pieces individually.

The existing cooling water discharge conduits will be extended by concrete flumes from the river bank across the canal wall.

Two sets of flap gates will be installed in the wall, one set to operate in each direction of flow. They will equalize the differential loading on the wall and will provide an alternate emergency source of cooling water. The operation of the project contemplates that in the event of hurrican tides, flow from the river downstream of the barrier will be continued so long as upstream inflow is low and the barrier pumps will maintain the upstream pool near 0.0 msl pumping the upstream inflow and cooling water discharge. The cooling water intake gates will be gradually throttled as the hurrican tide level increases. The flap gates will be set to open at 1 ft head differential to pro-

vide against sudden fluctuations in canal level which could arise in the regulation of the gate openings. If upstream inflow becomes large, the cool upstream inflow into the pool will maintain temperatures satisfactory for plant operation and the intake gate will be closed completely.

DESIGN PROBLEMS

The design problems associated with this site are not unprecedented, but are unusual in number and variety. The principal problems arise from the following conditions: (a) Location; (b) Hurrican storm tides and wave forces; (c) Tidal estuary flows; (d) Power plant cooling water supply; and (e) Foundations. Provisions for these conditions are described in the following paragraphs.

Location.—In the design of the project the interim report plan was considered as an authorization and a basis for developing a project for the conditions of the site in which the preliminary planning and studies were reviewed, continued, and supplemented.

At the initiation of design the survey report alignment was restudied. It was obvious that if the alignment could be straightened and moved a short distance upstream, the length of the relatively massive river structures could be decreased and the difficult construction at the points of land at the confluence of the Providence and the Seekonk Rivers could be eliminated. Since the preparation of the report, there have been developments that have made this change feasible.

On the east side of the river, the existing small industries are not a material factor in the establishment of a feasible alignment. At the west end of the project, an elevated expressway has just entered the construction phase. The embankment forms a satisfactory termination of the dike and permits a shortening of the alignment which is as a whole advantageous to the waterfront industries affected. The electric utility company occupies the west bank of the Providence River and is affected by an possible alignment in the area. The magnitude of the adjustments to assure no heavy damages make this plant the principal local factor in the selection of a feasible and economical alignment. This plant includes two separate power stations having a total capacity of 360,000 kw located 300 yd and 500 yd respectively, upstream of the river confluence with a coal storage yard immediately upstream of the confluence. An alternate fuel oil supply is maintained in tanks upstream of the coal storage yard.

In the past, coal has been transported by sea and unloaded at a dock downstream of the site. A conveyor belt paralleling and close to the river bank moves it to the storage area and reloads and moves it from storage to plant bunkers. Oil is discharged at a berth alongside the bank immediately above the river confluence. The report alinement is upstream of the coal-unloading dock and would not affect the existing coal-handling facilities, but access to the berth would be cut off. The oil discharge line could be extended downstream to a new berth at a relatively small expense.

Since the report was submitted, a self-unloading collier has come into service which is intrinsically more economical. The existing berth upstream of the river confluence is necessary for its satisfactory operation as it is designed to unload directly into the storage area. An alternate barrier alinement across the electric utility company yard upstream of the coal storage

area would allow full utilization of the new collier at the less consequential sacrifice of possible full future use of the yard in the vicinity of the barrier. The alternate alinement is at the practical shortest river width for optimum protection to the area and is much more economical to construct. It was therefore adopted for the final design of the barrier.

Hurricane Storm Tides and Wave Forces.—The determination of a grade adequate for protection against hurricane tides is the prime design consideration. This grade, termed the "still water" level, has been established by selecting a design storm and computing the resultant still-water level.

Most hurricanes which have affected the eastern coast of the United States have recurved after approaching the Florida coast, turning north. After recurving, the forward speed usually increases to 25 to 30 knots but may move as fast as 60 knots. As the winds of the hurricane approach the "eye" in a counterclockwise spiral the highest velocities occur to the right of the hurricane center where the wind and the forward movement are in the same direction.

The combination of winds and low barometric pressures, synchronized with tidal movements, produces a surge in the open ocean seldom exceeding 3 ft. It increases considerably on the continental shelf due to rising ocean bed and favorable shore contours. An additional rise in level occurs due to the effect of hurricane winds blowing over shallow coastal waters and bays, depending on the fetch, the depth of the water, and the intensity and duration of the winds.

In the September 1938 hurricane, the factors of storm path forward velocity, wind surge, and tide coincided to produce a still water level of 15.7 ft, msl in downtown Providence, R. I., which is about 8 ft above ground level.

Design Storm.—A design storm for use in determining the "still water level" for protective structures was established with the assistance of the U.S. Weather Bureau, Dept. of Commerce (USWB) and the Beach Erosion Board, aided by the Texas A. and M. Research Foundation. Possible future storms were considered in the light of past experience.

The design storm was based on the transposed Cape Hatteras hurricane of September 1944, having the greatest energy of any Atlantic coast hurricane, and adjusted by the USWB for the project. The 1944 storm was transposed so that it would be over water between Cape Hatteras and the New England coast, with a central pressure index of 27.71 in. near the mouth of Narragansett Bay. This barometric pressure is approximately 0.5 in. lower than actually occurred in September 1944. The center of the transposed hurricane was assumed to move along the critical storm track with various forward speeds. The critical storm path of the "eye" of the hurricane for the Narragansett Bay area is along the course of the Connecticut River.

Wind-field charts of the transposed September 1944 storm were prepared by the USWB for various rates of forward speed from 10 to 60 knots, and with different radii for the maximum winds. Two sets of charts were prepared for each of the design conditions; one set showing the hurrican at hourly intervals over the open ocean, and the second indicating the wind speeds at corresponding times in Narragansett Bay. In general, the faster moving storms, that is, 40 knots and greater would produce higher wind velocities offshore and in Narragansett Bay. The 50 and 60 knot storms would not produce wind velocities much higher than the 40 knot storms because such high forward speed has the effect of pinching off the warm moist air advection from the south and east, which, therefore, leads to some filling of the storm. The 50 or 60 knot storms are travelling so fast that they tend to run ahead of the tidal surge and

would not build up the surge as much as the 40-knot storm. The 40-knot storm, with large radii of maximum winds, was therefore selected as a basis for the critical conditions and computation of the surge heights at the mouth of Narragansett Bay and at the Fox Point Hurricane barrier.

Still Water Level.—Analytical computations of the storm surge variations at the mouth of Narragansett Bay for the project storm were carried out. The September 1938 storm surge was used as a basis for comparison. The wind fields for the 1938 storm and the transposed 1944 storm were used to determine the wind stresses over ocean waters and the resulting forced surge over the continental shelf.

The forward speed of the 1938 hurricane was computed to be about 42 knots from Cape Hatteras to the mouth of Narragansett Bay. The path of the center of the hurricane was about 80 miles west of Narragansett Bay thus producing a wind field most critical at the mouth of the bay. The wind-induced tidal surge arrived approximately 45 min before the normal high tide. With a coincident tide of 2.4 ft above msl at Newport Harbor, the maximum experienced still water elevation was 10.8 ft above msl.

The dynamic forces of the surge, funnelling up Narragansett Bay, the action of the hurricane winds on the water surface in the bay, and a coincident high tide of 3.1 ft, msl, increased the experienced still water level in the 1938 hurricane to the elevation of 15.7 ft at Providence.

Assuming that the design storm level would occur coincident with a spring high tide of 3.1 ft, msl, the still water level for the design storm was established at el 20.5 msl.

Hurricane Forces.—The size of the waves established the height of freeboard for dikes and walls. The magnitude of wave forces on various types of breakwaters is a key factor in deciding the type of construction on the land areas. Wave forces are a major factor in structural and foundation design.

In a "wave train" consisting of a series of waves, heights will vary considerably. The average height between crest and trough of the one-third higher waves is termed the "significant wave height." For the design condition of average maximum wind velocity of 90 mph and for the length of unobstructed "fetch" up the bay, a significant wave height of 6.5 ft was established. However, maximum wind velocity would precede the peak of the hurricane flood by about 1-1/2 hr. At the flood peak the significant wave height would be 5.1 ft.

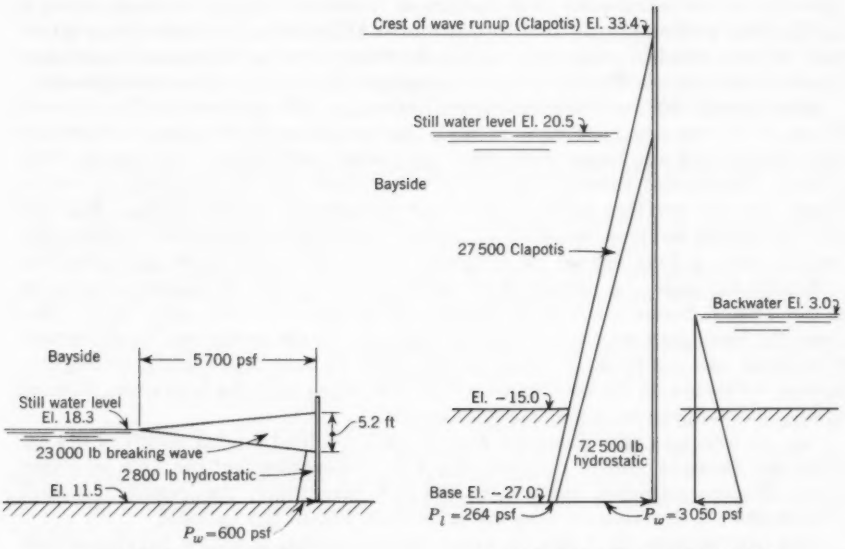
There are three general classifications of wave action against a structure, depending on foreshore conditions, of which two are possible at this project. When the water is relatively deep, waves will ride up the face of a structure, increasing the hydrostatic loading. On a sloping foreshore, waves will break when they reach a water depth of 1.3 times the wave height, delivering a heavy impact load from water and trapped air. The determination of wave forces for the various conditions are made from formulas developed by observations of experimental or experienced conditions.

The depth of water in the Providence River is in excess of 20 ft under design conditions, therefore only wave ride-up will occur.

The foreshores of the land sections of this project are flat. No studies of wave action for this condition have been made. It cannot be assured that the waves which can be supported by the water depth at the shore line will advance unbroken across the land to the dikes nor that they will break when they reach the dike. However, this possibility cannot be ignored. A vertical face such as a concrete wall would be subject to full impact forces. A massive section would be required to resist such forces, but the sloping rock face of a rock-

armored earth dike will effectively cushion wave action. An economical and safe earth dike structure can be produced by the selection of the proper size of rock for the wave height. In view of the great relative economy and practicability of rock-armored dikes, this type of structure is used on land areas except, naturally, for vehicular gates.

Diagrams of design hydrostatic loadings for non-breaking wave conditions and for breaking wave conditions are shown in Fig. 3 to illustrate the relative magnitude of the forces. Computations for the diagrams are omitted as they involve reference to voluminous tables.



(*a*) BREAKING WAVE FORCES (*b*) NON-BREAKING WAVE FORCES

FIG. 3.—WAVE FORCES

River Flow.—The Providence River is a tidal estuary with an upstream drainage area of 77 sq miles. The upstream area is drained by the Moshassuck and Woonasquatucket Rivers, which unite in downtown Providence River. There is upstream inflow and tidal ebb and flow typical to estuaries. In addition, the electric utility company uses a large volume of water for cooling purposes. The capacity of the river gates and pumps has been designed for upstream flood conditions with consideration to the effect of the barrier on normal flow conditions.

Experienced Storms.—The Providence River Basin is subject to three general types of storms that may be classified as continental, thunderstorm, and hurricane. The rapidly moving continental or cyclonic storms that cross the basin from the west or southwest produce frequent periods of rainfall but are not extremely severe. Continental storms are apt to be more critical when they are of the stationary frontal type which may produce appreciable

rainfall over a given area on several successive days. Thunderstorms may be of the frontal type associated with continental storms or of the local type and on a small drainage basin can produce high rainfall intensities. The most severe storms in the area have been of the hurricane type of tropical origin that move up the eastern seaboard. They are most likely to occur during the late summer and autumn months. The recent storms of September 1938, September 1944, August and September 1954, and August 1955 were of this type.

Heavy precipitation, often of torrential proportions, usually accompanies a hurricane and in some cases will arrive several days in advance. Pre-hurricane rainfall is produced when warm moist air, circulating around the eastern or northern side of a hurricane, collides with the cold air along a far-distant, preexisting front. The September 1938 storm, in which the greatest part of the rainfall occurred during the 4-day period before the hurricane crossed the coast of Connecticut, is an example of pre-hurricane precipitation. Approximately 90% of the total rainfall recorded during this storm at Providence, R. I. was pre-hurricane rainfall. Recent example of rainfall coincident with hurricanes are September 1944, September 1954 (Edna) and August 1955 (Diane). Hurricane rainfall has been responsible for the majority of record floods on the smaller river basins and tributaries in New England and has also produced serious flooding on major rivers when hurricane rainfall extended over a long period or followed a period of antecedent precipitation.

Runoff Records.—A review of the stream gaging and precipitation records at the United States Geological Survey USGS gage at Centerdale, R. I. indicates the basin has not experienced a major flood. Evidently the large amount of natural and artificial storage in the upper 24.5 sq miles has a strong retarding influence on the development of floods, especially the hurricane storms having short-duration, high-intensity rainfall.

As an example, the flood of August 1955 yielded a total runoff of 0.4 in. from an average basin rainfall of 8.4 in. Assuming that the 24.5 sq miles above Woonasquatucket Reservoir did not contribute, the resultant runoff represents 1.1 in. from 13.8 sq miles or only 13% of the rainfall.

The probability of a major river flood coincident with a hurricane tide is remote. The hurricanes of September 1938 and August 31,1954 (Carol) caused the greatest tidal damage in Providence in recent years, but in both storms, the total precipitation in the area was about 3 in. The reverse is true in the cases of the hurricanes of September 1954 (Edna) and August 19, 1955 (Diane). During these storms which caused the greatest river flooding, tides were only slightly above normal and caused no serious damages in Providence.

Pumping Requirements.—Although Providence has not experienced a major river flood coincident with a damaging hurricane tide within the period of record, the physical possibility exists with every future hurricane. Therefore, studies were made of various storms experienced in New England that could be considered as being associated with hurricanes.

The transposed September 1938 rainfall, rearranged in the most critical pattern would result in a floor at the junction of the Moshassuck and Woonasquatucket Rivers with a peak discharge of 10,000 cfs.

Basic conditions considered in the selection of pumps were peak rate of inflow, maximum tide conditions and pond storage capacity below damage stage in the Providence River.

The pumping capacity must be sufficient to provide for over-topping from wind and wave action during a design hurricane condition. Overtopping during the design tide extends over a period of 2½ hr and has a computed volume of 15 acre-ft and a peak rate of discharge of 354 cfs. No overtopping would be experienced in a recurrence of the September 1958 tide. Overtopping is obviously a minor factor in pumping requirements.

When the river gates are closed, intake of cooling water from downstream of the barrier will be continued until upstream inflow is high. The discharge will be pumped back through the barrier. As this operation will cease when inflow is high, pumping capacity requirements are not increased.

The Providence River pool upstream of the barrier, 30 acres in area, will be maintained at 0.0 msl as nearly as practicable, but will be drawn down to -3.0 to provide more pondage if a major upstream inflow is anticipated. This pondage will modify the peak inflow and reduce required pumping capacity.

The selected pump capacity of approximately 7,000 cfs at a static head of about 20 ft will be adequate to provide protection for the following combinations of tidal and river floodings:

1. All floods of record in the basin coincident with the proposed design tide at elevation 20.5 ft, msl.
2. The transposed September 1938 storm (17 in. of rainfall in 4 days) coincident with the September 1938 tide at elevation 15.7 ft, msl.
3. A flood comparable to the runoff from a 75-yr all-season rainfall, occurring in critical sequence with the proposed design tide of elevation 20.5 ft, msl.
4. A flood comparable to the runoff from a greater than 100-yr all-season rainfall, occurring in critical sequency with the maximum tide of record (15.7 ft, msl in September 1938).

The transposed 1938 storm coincident with the design tide would result in a maximum pool elevation of 9.7 ft msl, which is 3.7 ft above the beginning of damage stage. This stage would cause serious damage to downtown Providence but it is considered that the improbability of these coincident events precludes their use in design.

Gate Passage.—Passage must be provided in the barrier for normal flows with a minimum of change in existing conditions and for upriver floods with no increase in flood levels through the downstream area of Providence.

Three 40-ft wide gate passages, sill elevation -15.0, msl will adequately satisfy the criteria. The passages are located as close as practicable to the east bank to lessen recirculation of water warmed by cooling water discharge back to the cooling water intakes.

Normal tide ranges will not be affected except for a slight lag in time. The normal tide range is 4.5 ft.

The river gates must be adequate because a flood could reasonably occur at any time of the year. For this purpose a standard project flood for the total area was determined in accordance with established criteria. In this determination a rainfall excess of 9.12 in. was developed from 48 hr of rainfall of 12.0 in. with maximum losses of 0.1 in per hr. The peack flow for the total drainage area would be about 24,000 cfs. The coincidence of such a flood with a hurricane tide is too remote for consideration for pumping requirements. But the capacity of the gated openings through the barrier must be adequate to insure that the restriction would not contribute to damage during a major river flood with normal high tide conditions. The standard project

flood discharge would flow through the gate openings with a head of approximately 2.3 ft which would not increase upstream damages at normal high tides. Backwater computations indicate that the tidal portion of the Woonasquatucket and Moshassuck Rivers have a channel capacity at normal high tide of 6,000 cfs and 3,600 cfs, respectively. For greater discharges there could be overbank flow, but such flows would return to the Providence River above the hurricane barrier without being affected by backwater from the gates.

Navigation.—In the past, the Providence River upstream of the barrier was an active dock area and a dredged channel was maintained. With the disappearance of passenger and general cargo service, there is no navigation usage upstream of the barrier, slips have been filled, and the channel is silting. There is no thought or desire that navigation can or will be resumed in the river. In the event that silt deposits accumulate to such an extent as to require dredging because of interference with barrier or power plant operations, the width and vertical clearance of the gate openings will allow passage of small barges.

Some consideration was given to possible future development of the upstream pool as a marina, or as a hurricane refuge. With more desirable existing and potential installations downstream, there is no interest in the possibility of future use as a marina. The utilization of the pool as a hurricane shelter is unacceptable because of difficulty in timing gate closure to assure certainty of protection to the city while still allowing late arrivals access to the pool.

Cooling Water Requirements.—The currents in the Providence River are light and variable, having a maximum velocity of 0.3 knots under normal conditions. Current direction is generally upstream at the bottom and downstream at the surface. The river narrows and shallows towards the upper end, but averages 600 ft wide by 25 ft maximum depth abreast of the electric utility company's power plants.

Cooling Water Usage.—The river water for the plant condensers, intakes being near the bottom and discharges near the surface. The average demand of 850 cfs is the principal current-producing factor.

Cooling water is taken from the underlying stratum of sea water in the Narragansett Bay. The sea water at the mouth of the bay intrudes up the bay in the form of a wedge, tapering in thickness and decreasing in salinity towards the head of the bay. At the mouth of the Providence River the water at the bottom of the bay has been warmed slightly above temperatures at the ocean, but is entirely satisfactory for cooling purposes. By setting intakes as low as practicable, a generally upstream bottom flow of cool water is established.

The water is warmed about 15° passing through the condensers and is discharged near the surface where it is cooled by radiation and to some extent by intermixing with the underlying stratum. Upstream inflow is too small to effect much cooling. The proximity of intakes and discharges results in some recirculation of discharge water to the intakes. In the hotter summer months there is a marked rise in temperature of the cooling water from seasonal heating of the water and from plant operations.

River Temperature.—As the construction of the barrier will change the current pattern of the river, thereby affecting the cooling water, records of existing conditions are being maintained and will be continued through the construction period.

Extensive sampling was carried out in August-September 1959 to record current patterns and temperatures in the river. Data on power output and cooling water temperatures were furnished by the electric utility company. These showed temperature rise progressively up the estuary amounting to about 4° maximum and a marked built-up of temperature during weekday periods of high power output. River water temperatures immediately reflected cooler weather conditions near the end of the period.

Silting.—Silting has been continuous in the river within knowledgeable times, the river filling rapidly in the upper reaches and to a lesser extent below. Since the disuse of the navigation channel in 1940, as much as 15 ft of silt has deposited near Crawford Street at the head of the river where it has filled to low water elevation. As there is considerable organic content, it creates an odor nuisance in the summer season. Downstream, the Narragansett Electric Co. has dredged twice since 1950 to clear the cooling water intakes and to improve the channel.

Construction of the barrier may increase the rate of deposition slightly by slowing the currents.

Corrosive Elements.—Corrosive elements in the water have required the use of special alloys for the power plant condenser tubes. The river water contains about 16,000 ppm of sodium chloride as well as lesser amounts of sulphates and other mineral compounds from industrial and other wastes. The alloys are primarily selected for resistance to salt-water corrosion.

The upper 20-ft layer of silt deposits in the river bottom contains much decomposing organic material, trapping the hydrogen sulphide byproducts. During the recent construction of an expressway across the river above the electric utility company plant, the power company reports an increased rate of tube failures, which it ascribes to mixing of the silt into the water during excavation and subsequent entrance of contaminated silt and suspended sulphides into the condenser tubes.

Need for Cooling Water Canal.—Model tests to give an indication of the effect of the barrier on cooling water temperatures were conducted at the Waterways Experiment Station, Vicksburg, Miss. for construction and operational conditions. The tests showed that the barrier would impede normal circulation with the result that if no canal were provided, a temperature rise of about 3.4 F would result. Further, any cofferdams in the river would cause a similar temperature rise. They also showed that the canal would eliminate any such harmful effects and that somewhat cooler water than at present would be provided.

In order to assure the continuance of conditions equivalent to those now existent, a cooling water canal will be provided as the first step in the construction.

Construction of the barrier will require the removal of approximately 75,000 cu yd of silt from a location across the approaches to the cooling water intakes. The prior construction of the canal together with an extension past the excavation area will materially reduce the intake of silt and sulphides.

Geology of Site.—The presence in the foundations materials of thick beds of silt requires special provisions for foundations, embankments, and support of structures.

The river bed material across the full width of the Providence River is a dark gray organic silt ranging from 15 ft to 25 ft in thickness. At least the top 5 ft at midstream and even more toward the shore is a soft recent sedi-

ment. The remainder of the organic silt is stiffer and exhibits considerable shear strength for such materials.

Below the silt is a stratum of silty, sandy gravel and silty, gravelly sand, 10 ft to 15 ft in thickness existing in rather discontinuous layers of varying thicknesses.

Underlying the sand and gravel stratum is a deposit of inorganic silt, approximately 30 ft in thickness, randomly interbedded with layers of silt, fine sand and slightly plastic silty clay. These layers vary in thickness from 1/16 in. to 2 in., with the clayey portions generally occurring in the thinner layers. The stratum is of fairly uniform thickness across the width of the river,

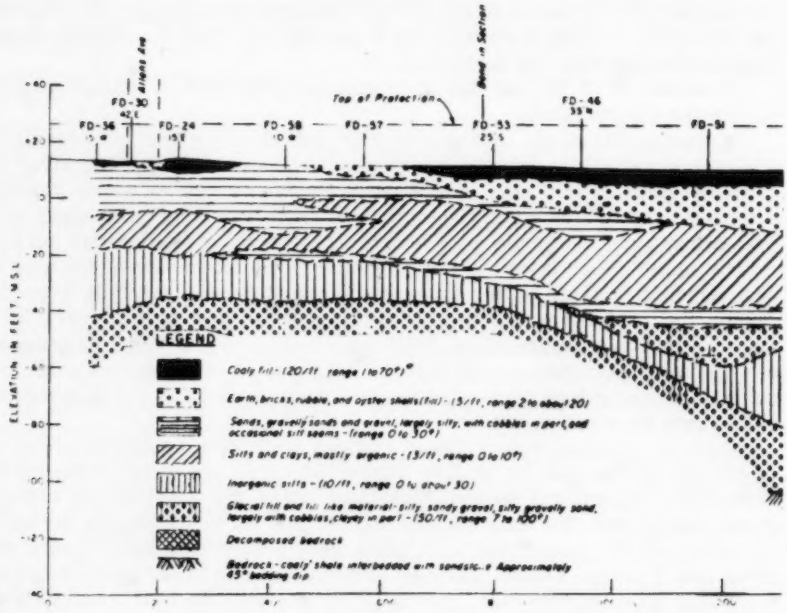

Top of Protection
FD-30
FD-56
FD-24
FD-58
FD-57
FD-53
FD-46
FD-54
ELEVATION IN FEET, MSL
LEGEND
Coaly fill - (20/ft range 1 to 70°)
Earth, bricks, rubble, and oyster shells (fill) - (5/ft, range 2 to about 20)
Sands, gravelly sands and gravel, largely silty, with cobbles in part; and occasional silt seams - (range 0 to 30°)
Silts and clays, mostly organic - (3/ft, range 0 to 10°)
Inorganic silts - (10/ft, range 0 to about 30)
Glacial till and till-like material - silty, sandy gravel, silty gravelly sand, largely with cobbles, clayey in part - (50/ft, range 7 to 100°)
Decomposed bedrock
Bedrock - coaly shale interbedded with sandstone. Approximately 45° bedding dip

FIG. 4.—GEOLOGIC SECTION ALONG APPROXIMATE

but appears to contain decreasing amounts of clayey-type materials in the vicinity of either shore.

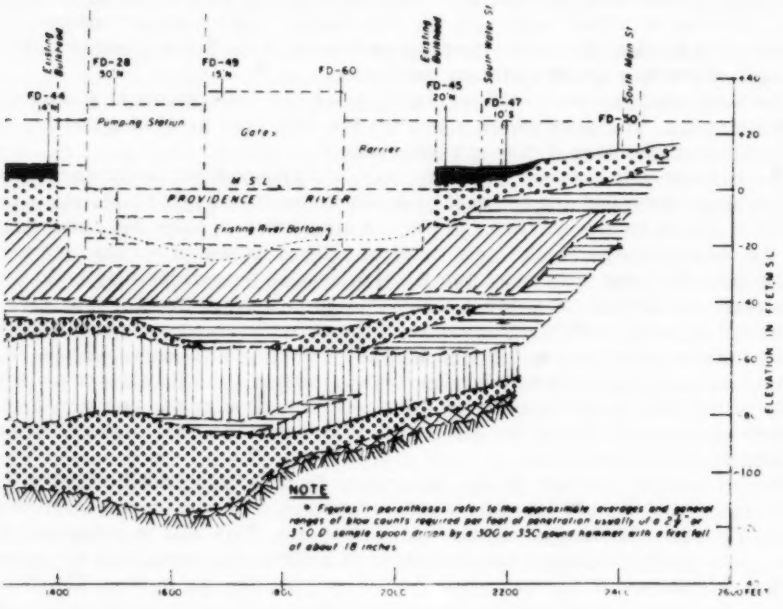
Beneath the inorganic silt zone at approximately 82 ft below mean sea level is a glacial till ranging from a silty gravelly sand to a silty clayey sand, both with cobbles. The thickness of the stratum under the west and central portions of the river is approximately 30 ft and diminishes to 10 ft below the east bank.

The underlying bedrock, which occurs at a depth of 110 ft at the west bank and midstream and rises to 85 ft below mean sea level at the east bank, is generally a black coaly shale interbedded with gray sandstones which dip at approximately 45°.

The present level ground surface on either bank is a result of gradual filling-in of the tidal flats for industrial use. The fill is an average of 20 ft in thickness and is composed of a mixture of silt, sand, and gravel, together with coal, ashes, brick, and shells. It has a wide variation in density and gradation, with shells and cinders found in pockets.

The silt and the sandy gravel layers found under the river thin out from the banks towards either end of the barrier and the glacial till rises in elevation. Fig. 4 shows the geologic section.

River Excavation and Backfill.—The bottom of the river structures are within the organic silt zone, some 10 ft to 15 ft above its lower boundary.



CENTERLINE OF BARRIER, LOOKING UPSTREAM

Thus, at least partial removal of the material is necessary. The possibility of leaving the remining silt in place and dumping a gravel working blanket over it, either before or after the driving of bearing piles, was considered. This would result in an appreciable savings, both in excavation and backfilling costs, but it was not considered feasible because the weight of the blanket would cause eventual settlement in the silt and permit a gap to develop beneath the bottom of the structure through which piping might develop should there be appreciable leakage through the cutoff sheeting. There would also be corrosive action from direct contact of the organic silt with the steel bearing piles and cutoff sheeting. Therefore, it is considered necessary that, even though

the structures are completely supported by the bearing piles, the organic silt be removed as completely as possible from beneath the foundations and replaced with granular fill. The excavated slopes will extend upstream and downstream at about the temporary angle of repose and will support the gravel and rock-blanket erosion protection material immediately adjacent to the finished structures.

It is estimated that excavation of the silt and the backfilling with sand "in the wet" would be more economical than attempting to do it within an unwanted cofferdam, inasmuch as heavy cellular or crib-type cofferdams would be necessary to balance the hydrostatic forces in the latter case.

The excavation can be done either in the open prior to erection of the construction cofferdam or subsequently within the cofferdam, at the option of the contractor. It is assumed that the excavation will be accomplished by clam shell buckets in either case, so the contractor's option would reduce to a matter of balancing the extra sheeting rental costs against any physical advantages of working from within an enclosure.

The excavated material will be placed in scows and dumped at a suitable disposal area. The absence of a nearby dumping area precludes the use of hydraulic dredges, even if they are otherwise economical.

The organic silt between the temporary cofferdam forming the canal extension at the west end and the east bank will be removed in a single stage or series of stages as the contractor prefers. A single wall open-ended cofferdam will be constructed into the east bank to allow removal of organic silt and keying into the bank with granular fill. Excavation operations may be in the open river or within cofferdams, whichever is more practicable. No cofferdams will be erected other than those required for later construction operations. In any case, excavation and backfill design limits shown will be required. The average bottom elevation of the silt is at 38 ft below mean sea level, which will be set initially as the depth of excavation for pay purposes. Pockets of inorganic silt below elevation -38 discovered during the excavation operations will be removed.

The remaining portion of silt excavation in the temporary cofferdam in the area to be occupied by the west connection, will be excavated entirely within an internally-braced single wall cofferdam. This will be scheduled at some time after the pumphouse structure is sufficiently completed to transfer lateral loading of the river bank to it by cross-bracing in the cofferdam. The cooling water flow will be channelled through the completed intake and canal before this cofferdam is installed.

Granular fill may be any natural granular material or a product of crushing, provided that no less than 30% by weight is retained on the No. 60 sieve and not more than 15% by weight passes the No. 200 sieve. It would be desirable if the material were completely free of fines, but studies show that it may not be economically feasible to specify an absolutely clean material. The contract specifications will require that the contractor be responsible for furnishing suitable granular material to meet the foregoing requirements. There are several acceptable developed and undeveloped sources of suitable material within reasonable hauling distance including a deposit in the bed of the Seekonk River 4 miles distant, which could be used in an all-marine operation.

A foundation sheet pile cutoff through the granular backfill is required because of the pervious nature of the fill, of the possibility of segregation of material due to the method of placement, and of the possibility of the settle-

ment away from the bottom of the structures. The cutoff will extend through the underlying sand-gravel stratum into the inorganic silt to obtain more positive foundation seepage control.

Steel sheet piling will be used. Steel piling is selected in preference to wood piling because of the possibility of hard driving in the sand gravel stratum.

Rock protection will be provided to the extent necessary to protect the foundation from erosion. The normal velocity through the river gates will be less than 1 fps. However, in a major flood severe scouring velocities will occur. A 15 ft width of riprap on bedding will be placed in the dry adjacent to the gates. Outside the limits of the cofferdams a 35-ft width of dumped rock, 3 ft in thickness, will be placed, in the wet. The maximum velocities at the pump intakes and discharges will be about 4 fps, therefore, a 3-ft dumped rock blanket, 35 ft wide, will be placed at these openings.

The granular backfill will key into the banks to assure the adequacy of the foundations immediately adjacent to the river backfill section (Fig. 5). The banks will be restored by placing fill which will necessarily fan over the organic silt outside the limits of the foundation fill. Before placing fill over the silt, the upper and softer portion will be removed. Consolidation of the remaining silt is anticipated when the fill is added. Stability analyses were made which indicate that the strength of the organic silt is adequate to support the added fill.

Cofferdams.—Although the details of cofferdamming schemes and the methods and sequence of construction will in general be left to the discretion of the contractor, certain definite restrictions or guide specifications must be imposed where the protection of existing property and the maintenance of established river usage by the abuttors of the project are concerned. The detailed designs of the cofferdam for the west abutment and the temporary canal bulkhead at the west side of the pumping station will be provided so as to assure protection of the existing west-shore area (Fig. 6). Likewise the inshore end of the east bank cofferdam and the scheme for protection of the existing utilities along South Water Street at the east shore will be detailed. As for the remainder of the construction schemes, the general sequence of operations will be prescribed as well as firm criteria for the layout and design of the cofferdam structures, such as soil design data and stress limits for the construction materials. It will be required that the contractor submit all proposed schemes in detail for approval prior to installation.

The blockage of the river will be limited to definite stages and will proceed in the following order to provide for adequate river and cooling water flow at all times:

1. Erect the permanent cooling canal wall parallel to the west shore and build a temporary extension downstream of the barrier alinement. This is to shield the inflowing cooling water from the area where excavation will be in progress.
2. Erect a cofferdam around the pumping station and proceed with its construction.
3. Erect a separate cofferdam around the river gates and east barrier section. (This may be done simultaneously with the pumping station.)
4. After the river gates are completed to the extent that full river flow can be accommodated, the remaining center barrier section will be constructed.

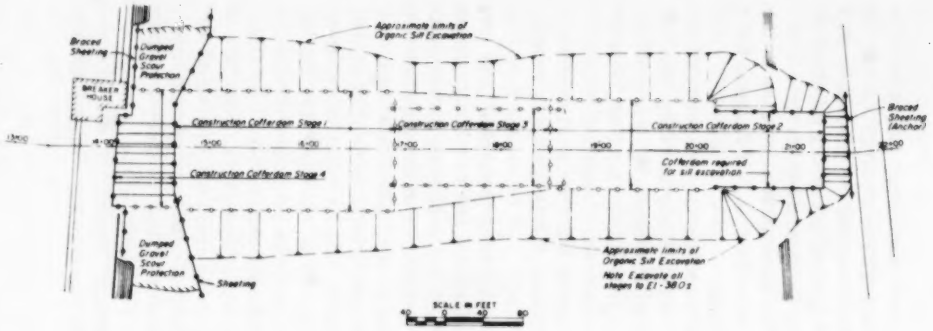


FIG. 5.—RIVER EXCAVATION SCHEME

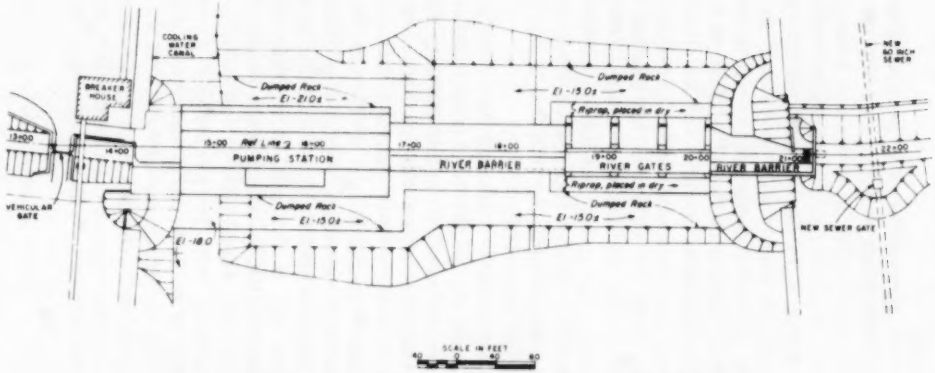


FIG. 6.—RIVER BOTTOM AND SHORE PROTECTION SCHEMES

5. After the pumping station sub-structure is essentially completed, the cooling water canal flow will pass through the intakes and the cofferdam for the west connection will be installed. The heavy lateral thrust from the earth forces at the shore will be transmitted to the intake structure. This cofferdam will be designed in detail in the contract drawings.

Two basic cofferdamming schemes have been considered.

Cellular Cofferdam.—A series of 45-ft-diam sandfilled cofferdam cells would encircle the structures for the required stages, except that due to lack of space, single-wall braced sheeting would be required at the end adjacent to the cooling canal and possibly at the ends facing the center barrier section. Excavation and backfill would be accomplished in the wet, after which the cofferdams would be dewatered by open pumping or with wellpoints if necessary. The piles would be driven and the structures completed in the dry. The inside line of the cells would provide for a clearance of 15 ft to 20 ft to permit space for driving batter piles and placing riprap. There would be no internal bracing present to interfere with construction.

Single-Wall Cofferdam.—The cofferdams could be erected before excavation and replacement of the organic silt and driving the foundation piles but would more than likely be constructed after the dredging, backfilling, and pile driving has been accomplished in the open. The width of the cofferdam would vary according to the amount of clearance deemed necessary for the pile driving and forming of the structures as well as for placing the riprap.

Prior to unwatering the cofferdams, one tier of internal bracing would be installed or possibly a row of exterior battered tension piles would be driven to give lateral support at the top of the sheeting. The cofferdams could then be unwatered in stages with additonal tiers of internal bracing. The structures would then be constructed. It is possible that the cofferdams could be safely unwatered by open pumping from sumps, if the cofferdam sheeting penetrates into the inorganic silt stratum. In order that no detrimental movement of the foundation fill would occur during the dewatering period, and also to eliminate a tier of internal bracing, a tremie seal extending over the area within the cofferdam will be permitted. The foundation piles would assist the tremie slab in resisting the hydrostatic uplift. The tensile load on the foundation piles would be limited to 30 tons or to a value selected after load tests are performed. The piles could be cut to required grade above the tremie slab after unwatering the cofferdams.

If a tremie slab is selected by the contractor it will be required that the surface of the tremie concrete be kept below the structural base slab of the structures. It would be economical and feasible, however, to permit the tremie slab to form the lower portion of the massive central barrier section. The use of a tremie slab or other methods by the contractor will be controlled so as not to affect the basic design or performance of the structures.

Structure Foundations.—The weak foundation materials require that special measures be undertaken for support of structures. Three possible measures for support of structures in the river area were considered.

1. The use of compacted or wetplaced fill or a thick tremie mat to reach firm bearing material, without piles.

2. Short compaction piles (27 ft more or less) driven through backfill replacing organic silt and extending into the underlying sand and gravel stratum.

3. Long bearing pile (85 ft more or less) driven into the glacial till overlying the bedrock. This scheme is adopted herein.

Direct Bearing Without Piles.—The boring samples from the sand and gravel stratum underlying the organic silt indicate that the material is, for the most part, well-graded gravelly sand or sandy gravel at least 10 ft in thickness. The foundation soils below the top of the existing sand and gravel stratum are adequate, in strength to insure no foundation failure by shear. However, settlement would occur due to the consolidation of the inorganic silt stratum. The computed settlement is in the order of 3 in. The consolidation test data show that consolidation would occur rapidly as the load is applied, and, in view of the stratified nature of the deposit, it can be assumed that the estimated settlement would take place within a short period after application of the structure loads, and would progress along with the construction.

One method considered for achieving direct bearing was by careful and complete excavation of the organic silt down to the sand-gravel stratum and replacement with granualar backfill in the dry within a dewatered cofferdam. There would thus be a total of 25 ft of granular material below the pumphouse, and even greater thicknesses below the other structures.

A second method would be to pour a tremie mat, approximately 13 ft to 15 ft thick, on the sand-gravel stratum after excavation of the soft organic silt. This would cause slightly greater settlement due to consolidation within the inorganic silt zone, but within permissible limits.

A third method considered was that of excavating the organic silt and backfilling with coarse granular or crushed material in the wet. There would be negligible settlement within the fill if clean coarse granular material was used.

Of the aforementioned schemes, the third would be the most economical.

Short Piles into the Sand-Gravel Stratum.—For compaction piling into the sand-gravel stratum both timber and precast concrete piles were considered because they would compact the wet-placed granular fill during driving. The timber piles would be untreated since they would be continuously submerged. They would be driven partly into the sand and gravel stratum where it is anticipated that they would "take up" and provide a design capacity of 24 tons each. Since the structure loads would then be transmitted to near the pile tips, a settlement of approximately 3 in., due to consolidation in the inorganic silt zone was computed. This would take place rapidly during construction and would be of no serious consequence.

Because of the possibility that a higher capacity pile (concrete), more widely spaced, might punch through the sand-gravel stratum, the wood pile was favored. The design would include provision for a temporary overstress of 25% (30 tons total) for loadings during construction periods and extreme hurricane conditions.

Long Bearing Piling.—The preceding schemes were discarded in favor of long bearing piles driven into the underlying glacial till, requiring piles some 75 ft to 85 ft in length. While the predicted settlement due to the consolidation characteristics of the inorganic silt would be quite tolerable, there remains the possibility of later additional settlement created by vibratory forces, such as might be set up by pounding waves, pumps or industrial operations on shore. Upon examination of freshly-opened samples of the interbedded inorganic silt, the soil appears quite sensitive to vibration but the magnitude, if any, of actual subsidence of the completed structures which might be attributed to this cause is indeterminate. In view of the unknown effects of vibration on the silt layer, and the lack of conclusive data to prove

that such vibration would not be detrimental, the decision was made to adopt a foundation design in which long bearing piles are driven into the glacial till deposit and to the bedrock, if necessary, for end bearing.

A non-displacement type pile is selected in view of the possible disturbance of existing structures on shore through excessive vibration in the inorganic silt zone. A displacement type pile of precast concrete could possibly be advanced with the aid of pre-excavation by jetting, but in view of the large percentage of batter piles, and the expectation that the holes would require casing through the upper granular materials, it is not considered feasible.

The pile selected is a 14-in. steel H-pile section weighing 89 lb per linear ft. Since the piles would be driven to virtual refusal to the bedrock, or into the till immediately above bedrock, a high capacity is justified.

The possible corrosion of the steel piling due to such sources as stray electric currents, electrochemical reactions between various soil types, corrosive groundwater or direct contact with possibly corrosive soils such as the organic silt has been considered. The possibility of consequential corrosion is not considered serious, however, and no special protective measures are provided. The recommendations of the recently revised Boston Building Code (Boston, Mass.) are being followed in the adoptiong of loading criteria. This provides for a pile design capacity of 7,500 psi of cross section. Thus, the normal design load for the 14-in., 89-lb section would be nearly 90 tons, the figure which is adopted for this design. A compression load of 135 tons, a 50% overstress, can be tolerated for hurricane conditions, in which the maximum design loads are infrequent and would be dynamic in nature. The 135 ton value can also be used for the temporary construction condition.

For those structures in which it becomes economical and practical to reply on the tension value of batter piles to help resist the large lateral forces during storm conditions, a value of 30 tons per pile axially is adopted. This is based on an average shearing resistance of 125 psf of pile surface.

Pilings are battered, part in a downstream direction and part in an upstream direction. The downstream battering is required for lateral hydrostatic loading conditions. For normal conditions, the vertical loading is distributed to both batter systems. The tension value of the upstream battered piles is utilized in analysis for storm condition loading to reduce the number of pilings. Analysis has been made according to the "elastic center" method.

Foundation conditions under structures in the land areas likewise require the use of support piling. Steel H-piling will be used under the vehicular gates and concrete walls and will be battered for lateral loads. Sewers and electrical conduits passing under the dike or the gates will be supported on timber piling.

Dike Foundations.—As stated previously, the silt layers which overlie glacial till, extend under man-made fill material on both banks. The fill materials are assumed to be highly pervious due to random pockets of shells, cinders and other debris. Presence of unburned coal and cinders near the surface is a factor which makes the use of steel sheeting unfavorable in considering the construction of a foundation cutoff for seepage control.

The dike will settle as a result of the consolidation of the organic silt stratum immediately underlying the surface fill. Settlement as a result of consolidation of lower soil strata will not be significant due to their depth and the characteristics of the materials.

Special consideration must be given to the control of seepage at the junction of the dike embankment with concrete structures. Differential settlement

between the embankment and the pile-supported structures could open up cracks at the face of the structures, therefore these faces will generally be battered and a seepage fin will be provided. Preloading of the foundation will be carried out wherever feasible to reduce differential settlement and minimize possibility of development of tension cracks.

The stability of the dike foundation is amply adequate for anticipated conditions of flood "drawdown" and for the added loading on the foundation.

Computations of settlement due to consolidation of the organic silt stratum under load indicate a maximum settlement of 20 in. This figure is corroborated to some extent by recorded settlement of the 180-ft diam oil storage tank near Station 7 + 00. It is assumed from soils data that 50% of the settlement will occur during construction of the dike. Grades can be adjusted for the estimated foundation consolidation. Nearby structures are old brick buildings,

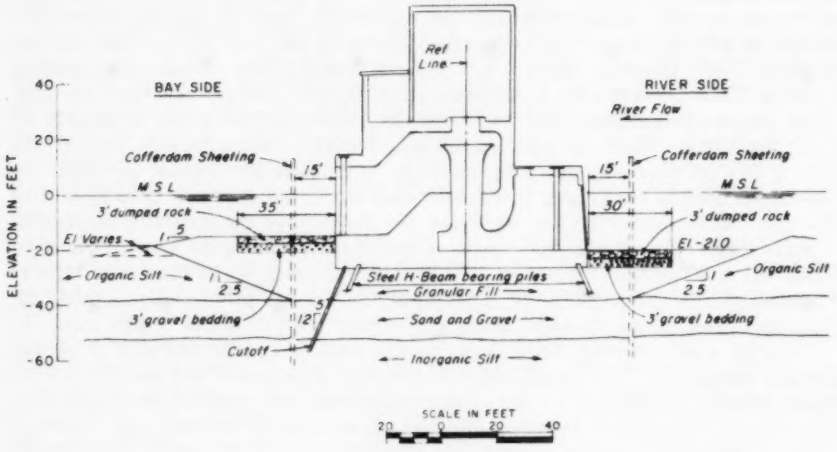


FIG. 7.—TYPICAL SECTION AT PUMP STATION

apparently without pile foundations. The influence of settlement under the dike is expected to result in slight and inconsequential cracking. A check on elevations and conditions at the brick buildings and also at the oil tank and pile supported breaker house of the electric utility company will be maintained during construction.

Because of anticipated settlement and of the deep cutoff, all existing utilities will be relocated through the gate structures, except for two trunk sewers and an electric duct line. These will be supported on piling and a sheeted cutoff provided beneath them in case of settlement of the foundation below them.

Structural Design.—Structural design generally follows standard procedures. There are some special considerations and provisions which will be noted.

Normal allowable concrete and structural steel stresses were increased 33-1/3% for dynamic wave loadings because of the infrequency of full load application.

Type V cement with a dense mix will be used in the concrete due to the high sulphate content of the Providence River water. Local practice often provides granite facings with lead-caulked joints where normally immersed in water, but this precaution would be costly and was considered unnecessary.

Where breaking wave forces are possible, vertical concrete faces have been eliminated so far as possible. On the west abutment of the river the rock armored face of the dike on the bay side has been continued to the river, although the back slope has necessarily been replaced by a retaining wall to gain space for an access roadway. On the east abutment the barrier wall has been extended into the bank area and the dike has been wrapped around it to absorb wave impact so far as possible.

The bases of the pumping station and the river gates are integral units to minimize differential movements within the structure. Temperature changes after construction will be negligible as the bases will be immersed in water of relatively constant temperature. In the pumping station, contraction joints will divide the structure above the base into four monoliths of 66 ft maximum length and 91 ft maximum depth (Fig. 7). In the river gates, expansion seats will be provided for the access bridge across the top of the piers.

Maintenance and Operation.—The barrier will be maintained and operated by the city of Providence.

The most important provisions in maintenance procedure will require that the pumps will be check-operated at off-peak power load periods of the electric utility company and that not more than one pump will be operated at a time. This is necessary because of the magnitude of the power requirements of the pumps. Scheduling the operation at off-peak periods is advantage as no additional plant capacity is required and the normal demand charge can be omitted.

The operation of the barrier in the imminence of hurricanes will produce much inconvenience, as the heavy volume of traffic on Allens Avenue will be diverted, sewers will be backed up, and at least one large pump must be operated intermittently. Gate closings must be directed to make certain the protection of the city at reasonably minimized inconvenience. A close coordination with USWB must be established to follow the predicted course and intensity of the storm. Operational procedures have been given general consideration and will be studied and prescribed at a subsequent date.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

GREAT VOLGA WATERWAY

By Otakar W. Kabelac,[1] F. ASCE

SYNOPSIS

This paper surveys the engineering development of the longest and largest (with respect to draingage area) river on the European continent from unregulated natural conditions into a modern waterway system having direct access to all major Russian seas. The Volga Project is the first to come under the Soviet hydrologic planning program for the development of water resources, and was begun in 1926. Besides navigation, the project is designed for hydroelectric power production, water supply, and soil rehabilitation.

INTRODUCTION

The opening of the Volga-Don Canal in June 1952 marks an important turning point in the inland transportation of the USSR. While adding the "missing link" to the Great Volga system, it initiated a superarterial waterway across the European part of the USSR and provided the junction with all major Russian seas. The engineering of the Volga River includes the largest accomplished hydroelectric power, industrial, and agricultural water supply combine in the Soviet Union. The recently accelerated Volga flow decline, which coincides with the construction activities, indicates hydrologic weakness of the project.

MOSCOW, PORT OF FIVE SEAS

The Volga-Don Canal, known officially as the Lenin Canal, when opened for operation, establishes a long-sought outlet of the landlocked Caspian Sea

Note.—Discussion open until July 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 1, February, 1961.

[1] Army Map Service, Corps of Engrs., U. S. Army, Washington, D. C.

FIG. 1.—ENTRANCE OF THE CANAL AT KALACH, LOCK NO. 13

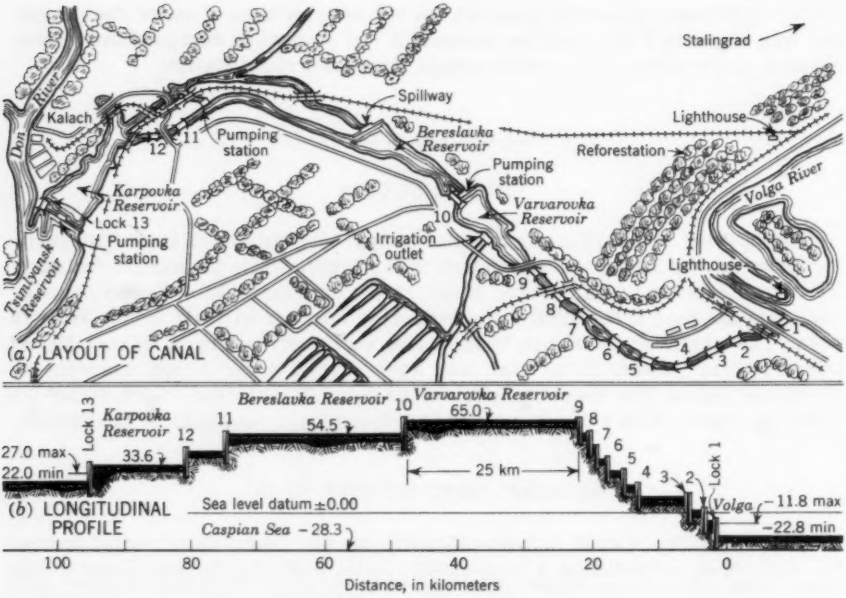
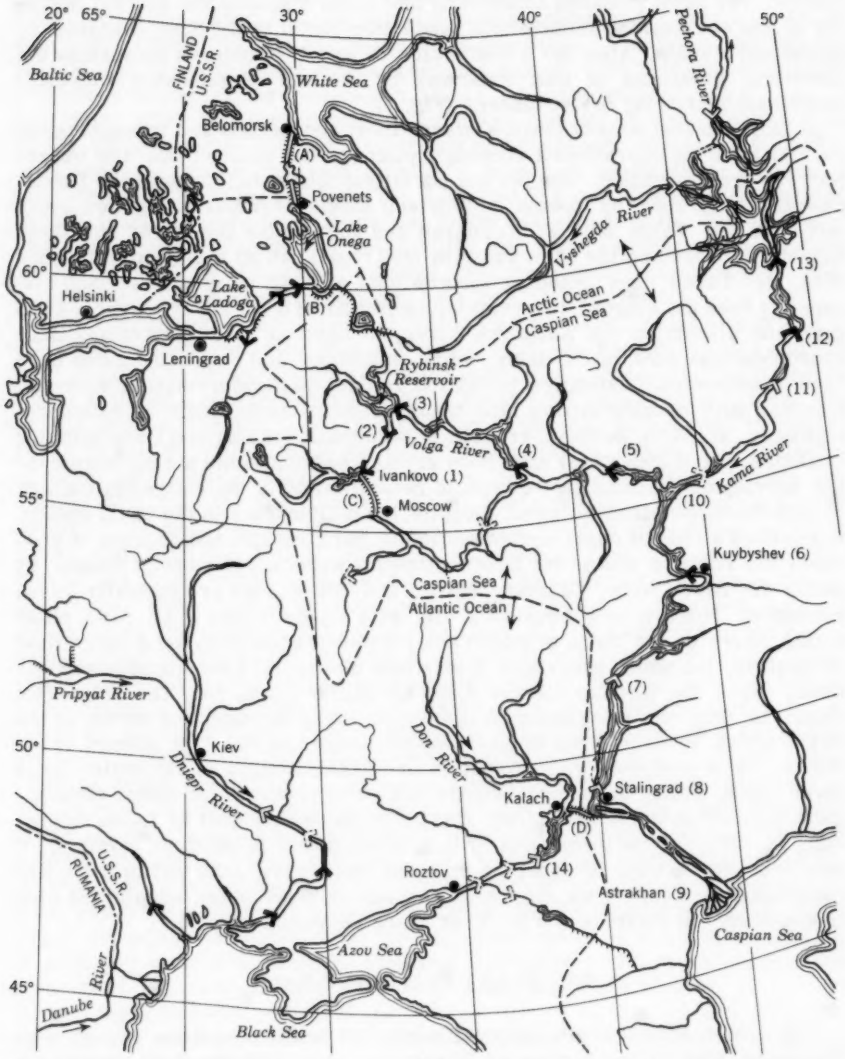


FIG. 2.—VOLGA - DON (OR LENIN) CANAL



Hydraulic centers:

(1) Ivankovo
(2) Uglich
(3) Cherbakov-Rybinsk
(4) Gorkiy
(5) Cheboksary
(6) Kuybyshev
(7) Saratov
(8) Stalingrad
(9) Nizhne Volzhskaya
(10) Nizhne Kamskaya
(11) Votkinsk
(12) Molotov-Perm
(13) Solikamsk
(14) Tsimlyansk

Major navigation canals:

(A) White Sea
(B) Mariinsk system
(C) Moscow-Volga
(D) Volga-Don

Major watershed
International boundary

Dams:

In operation
Under construction
Proposed

0 400 800
Scale in kilometers

FIG. 3.—THE GREAT VOLGA WATERWAY

toward the open Black-Azov Sea, by the use of the Don River junction at Roztov. The news releases concerning that event stressed the epochal meaning of this engineering achievement, and compared it to the Suez, Panama, or Mittelland Canals. Also the architecture of canal structures emphasize the historical greatness of this waterway by the use of extensive sculptural adornment glorifying Soviet leaders (Fig. 1).

It is of interest to note that the international economists and transportation experts have not expressed a very high opinion on the value of this new waterway for transportation. The 101 km (63 miles) long canal crosses the Black-Caspian watershed by means of thirteen single-chamber navigation locks (nine on the Volga side at Stalingrad and four on the Don River side from Kalach) to overcome the 88 m and 44 m (290 ft and 145 ft) elevation difference (Fig. 2). Three huge pumping installations with 45 cu m per sec (160 cfs) capacity feed the summit reservoir with water lifted from the Don over a 44 m head and also from the Karpovka River. In addition, a complex of auxiliary hydrotechnical structures, dams, weirs, spillways, and regulating gates have been constructed. Extensive irrigation reclamation reforestation systems, together with comprehensive land transportation media (such as railroads, highways, airports, bridges, crossings, terminals, loading and train shipping facilities, and warehouses) have been erected and constitute a civil engineering development and unified economic rehabilitation of the entire Stalingrad-Kalach-Roztov-Astrakhan area. Despite the deficiencies in the canal design, it assumed a role of considerable economic and strategic importance: it provided the missing link in the intercontinental waterway system of Russia, by which the Black-Azov, Caspian, Baltic, and White Seas are mutually interconnected. Moscow thus became a city with a port to five seas. Also, other major cities of the Union acquired sea communication (Fig. 3). A large fleet of vessels, including passenger liners with dining and sleeping accommodations, serve the Moscow-Roztov 3,500 km (2,200 miles) run. The Volga-Don Canal is only one component of the Volga River development known as the Great Volga Project. This project was introduced as the first attempt of the Soviet for a complete reconstruction and rehabilitation of the entire large river basin, within the comprehensive economic program for global development of Russian water resources. This program was initiated by Lenin originally; in 1921, for the purpose of hydroelectric power production. However, it was widened in 1926 in order to embrace waterways, land melioration, and rehabilitation as well as industrial and agricultural water supply; the plan was executed in a series of Five Year Plans beginning in 1928.[2]

THE GREAT VOLGA PROJECT

The Volga River, the traditional migratory route of nations, occupies an important role in the history of Russia, not only because of its strategical location between two continents, but mostly as a medium of East-West communication. The largest river of Europe, though only the fifth among the Russian rivers, the Volga, 3,700 km (2,300 miles) long, drains the major part of Soviet territory west of the Ural Mountains (1,480,000 sq km), which includes Moscow. The Volga has more than 1,000 tributaries, which (together with the main stem) represent, even in unregulated conditions, a navigable system of

[2] "Die Verkehrstechnische und Kraftwirtschaftliche Erschliessung der Volga," by K. A. Pohl, Die Deutsche Wasserwirtschaft, Stuttgart, 1955.

20,000 km (12,500 miles). The usefulness of the Volga as a medium of waterway transportation was greatly impaired, in the past, because of its natural outlet into the Caspian Sea, the world's largest inland body of water which is depressed (1960) 28.30 m below the world sea datum. The Volga, for 800 km (500 miles) flows at minus stages, reaching minus 13.50 m at Stalingrad, the present junction of the Volga-Don Canal, and descends toward the sea at Astrakhan after bifurcation into an auxiliary channel at Akhtuba. The delta of the Volga extends 170 km (105 miles) upstream and consists of 80 major branches and more than 400 auxiliary trenches which cover an area of 134,000 sq km (52,000 sq miles). The delta is 180 km (110 miles) wide having shrunk from the previous 220 km, due to the regression of the Caspian Sea.[3] The geomorphologic formation of the Volga bed offers an outstanding example of the influence of terrestrial rotation. The first observation and measurements of this phenomenon were made here by E. K. von Baer.[4]

The Volga reconstruction plans of 1926 were initiated following World War I, when the extreme deterioration of Russian railroads became evident and the near collapse of the entire transportation system was imminent. The Supreme Soviet ordered a comprehensive study of these conditions, which included the reevaluation of the transportation capacity of Russian waterways. The study revealed that the Volga, in its natural unregulated state, despite the limitation imposed by seasonal influences (periods or floods and frost) and the low loading facilities for the primitive Volga boats, carried an average transportation volume corresponding to that of a 6-track first class railroad. It was estimated that through hydrologic planning, engineering, and canalization of the Volga channel, it could be built into a carrier with a capacity of 40 tracks, when proper correlation with the existing and planned railroad net is taken into account. In view of this study, the VII Congress of the Communist Party approved the decision of the Supreme Soviet to retain the Volga as the hub of the soviet transportation system west of the Ural and ordered its reconstruction into a superarterial waterway across the European part of USSR. By means of auxiliary canals and waterways connecting the Atlantic and the Arctic Oceans, the Volga system was planned and designed to establish the interconnection among the major Russian seas, the Baltic-White, Black-Azov, and Caspian. G. Riesenkampf was then assigned to prepare the hydrologic part of the project, based on the principle of "maximum use of rivers as optimum solution of complex fluvial problems," later known as "maximalization of the rivers and river basins," by V. E. Timonoff, Stalin's principal advisor on waterways.[5] The Volga project embraced a global development of the water supply for inland navigation hydroelectric power production and rehabilitation of arid and semi-arid steppes, including the transfer of large water masses from the Arctic drainage to the Caspian Sea, to arrest its sinking.

Structural Standards of the Great Volga.— The project provides for a transformation of the main stem of the Volga, Kana, and the Don Rivers, by means of big dams, termed "Gidrouzel" (Hydraulic Center) into a succession of huge artificial lakes (Vodokhranilishe) that raise the river level (thus creating hydraulic heads) and retain the excess of the spring floodwaters behind them. Because of low gradient (an average of 0.0003), the dams are proportionally

[3] "Gidrografiia SSR. (Vody Sushi)," by L. K. Davydov, Leningrad, 1955.

[4] "Rivers under Influence of Terrestrial Rotation," by O. W. Kabelac, Proceedings, ASCE, Vol. 83, No. WW 1, April, 1957.

[5] "Role of Inland Waterways in the General Transportation Network of a Country," by V. E. Timonoff, XVIth Internatl. Congress of Navigation, Brussels, 1935.

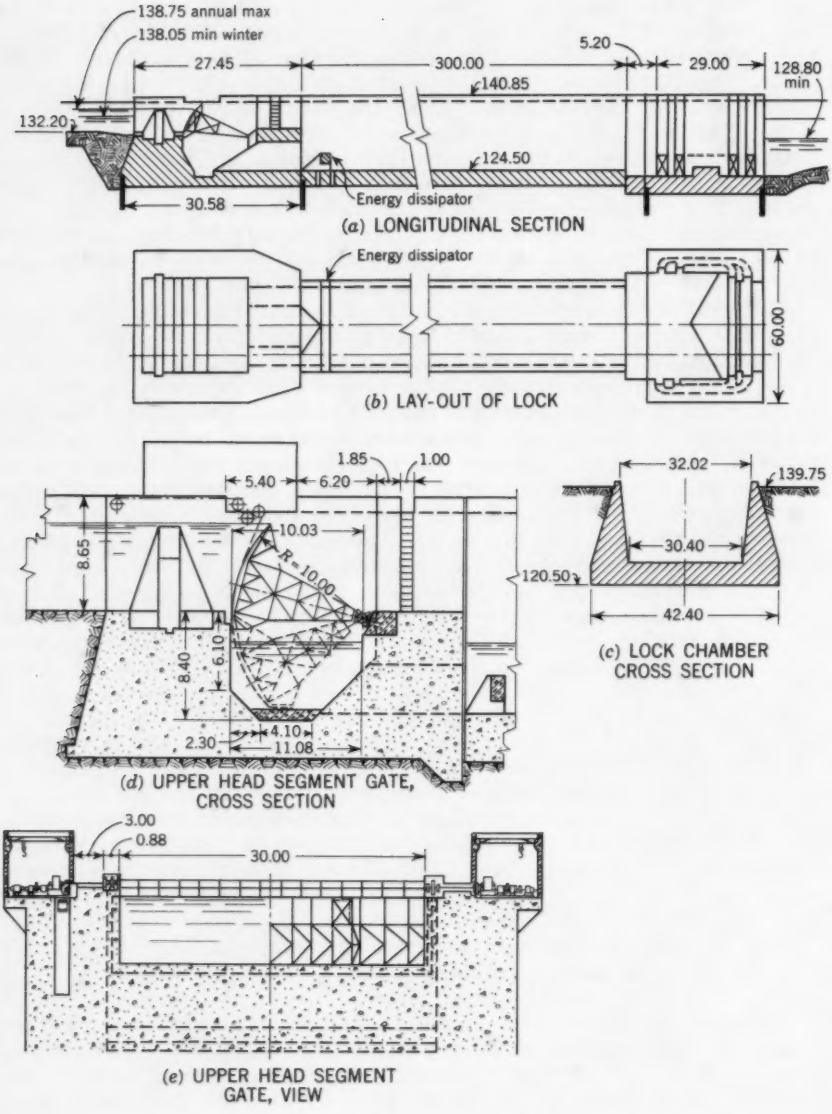
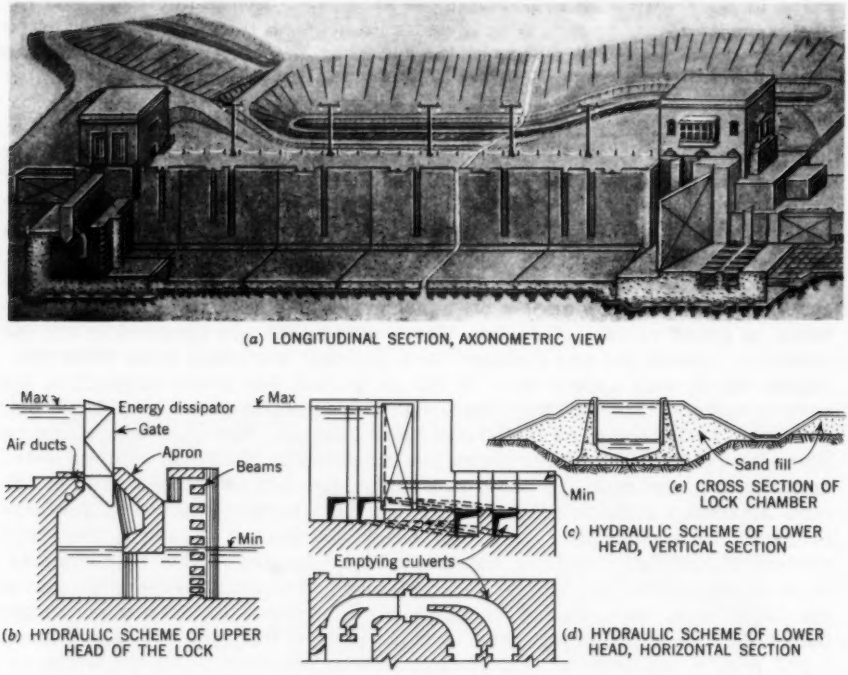


FIG. 4.—LOCK NO. 3 OF THE MOSCOW - VOLGA CANAL, A TYPICAL NAVIGATION LOCK OF THE GREAT VOLGA WATERWAY

low earth gravity structures of oversized dimensions which reach in some instances 12.8 km (9.5 miles) in length. The basic heavy silt and detritus carried by the river, enriched by clay and seepage-proof material, is used as the basic material of construction. The material is shaped into a dam structure by use of huge centrifugal slush pumps, following a method especially designed for this purpose by Soviet engineers.[6]

The retaining reservoirs, called sometimes "sea" or "lake," are parts of the waterway by which Riesenkampf intended to duplicate the natural conditions of Russian rivers and lakes in the North, such as Lake Ladoga and



(a) LONGITUDINAL SECTION, AXONOMETRIC VIEW

(b) HYDRAULIC SCHEME OF UPPER HEAD OF THE LOCK

(c) HYDRAULIC SCHEME OF LOWER HEAD, VERTICAL SECTION

(d) HYDRAULIC SCHEME OF LOWER HEAD, HORIZONTAL SECTION

(e) CROSS SECTION OF LOCK CHAMBER

FIG. 5.—TYPICAL NAVIGATION LOCK OF THE VOLGA - DON CANAL

Oniega. The first of the large Volga reservoirs, accomplished in 1941, the Rybinsk-Tscherbakov, which covers 3,750 sq km (1,850 sq miles) and compounds 24.8 cu km (20×10^6 acre-ft) was designed primarily for the improvement of the Volga navigation conditions. It may supply 600 to 800 cu m per sec (21,000 to 28,000 cfs) and ensure a minimum navigation depth of 2.60 m (8.5 ft) all the way to Astrakhan (prior to the construction of Kuybyshev and other structures built since 1945). The Kuybyshev "sea" extends for a distance of 600 km (370 miles), has an area of 9,500 sq km (3,700 sq miles), and retains

[6] "Volgo-Don, Obshee Opisanie Sooruzhenii," edited by S. IA. Zhuk, Moscow, 1957.

38 cu km (31 x 10^6 acre-ft). Tsimlyansk reservoir on the Don, 280 km (170 miles) long, in parts 30 km (18 miles) wide and of 12.8 cu km (10.5 x 10^6 acre-ft) retaining capacity, serves as supply storage for irrigation systems of the southern steppes in the pre-Caspian regions.

The navigation standards of the Great Volga are regulated by the Supreme Soviet decree of 1926, which ordered a minimum of 5.00 m (16.5 ft) depth of navigation canal in the ultimate state of development, accessible for vessels 4.50 m (15 ft) draft. The large Volga boats (200 m by 30 m) were used to indicate the minimum horizontal measurements required for locks, canals, harbors, and other navigation structures. The standard Volga locks have chambers 300 m (950 ft) long, 30.40 m (100 ft) wide at the bottom, tapering 1:100, and gates of 30.00 m clearance. The upper head has a 5.00 m minimum vertical clearance at the sill, and is equipped with segment or vertical roller gates opening downward. The filling of the chamber is over the crest of the gate while lowered in the open position (Fig. 4). The lower gates are of the miter type.[7]

The Volga-Don Canal locks (Fig. 5) are smaller: 185 m by 18 m by 4.5 m (600 ft by 60 ft by 15 ft). The original plan called for standard-size locks; however, when the plan was changed and the feeding of the Volga-Don Canal by gravity from the Don River was abondoned in order to save the water for irrigation (and also to save earth movement), the locks were restricted in size. This change greatly diminished the transportation capacity of the canal.

The Hydroelectric Cascade of the Volga.—The hydraulic centers (Figs. 6 and 7) regulate the flow across the dams by movable weir and spillway structures of great variety of design. These structures are operated by remote automatic control and are designed for a probable maximum flood water discharge which may appear in a 10,000 yr period. For power production, the soviet engineers adopted the "river-run" hydroelectric power plant principle, developed in Germany and introduced there at the Kachlet Dam on the Danube River in 1926. These power plants are structurally combined with the weir-spillway concrete masonry block as a part of the dam and placed so as to secure maximum utilization of the flow and high turbine efficiency. The low-pressure turbogenerator units may reach 200,000 kw capacity. The Volga system of hydraulic centers, together with the Kama and Don developments, created (as of 1960) the largest accomplished hydroelectric power combine in the USSR with an installed capacity in the order of 10,000 mw and annual output of 100 x 10^6 mwh. This capacity is to be increased by an additional 1,000 mw by the transfer of parts of Petchora and Vytschegda Rivers south, with the accomplishment planned in 1965. The present Volga hydroelectric power combine consists of eight existing hydraulic centers, with three centers under construction and an additional three proposed (Table 1).[8]

Auxiliary Waterways of the Great Volga.—The basic purpose of the Volga project was to establish inland waterway connections among the major Russian seas, and (for prestige reasons) to develop Moscow as a great seaport of the union. This was accomplished, in principle, in 1952 by the Volga-Don Canal and the following auxiliary canal junctions (Figs 3 and 8):

a. White Sea Canal, a 227 km (140 mile) long waterway between Povenetz on the Lake Oniega and Belomorsk (Soroka) at the White Sea. It has 19 locks, overcoming an elevation difference of 75 m (246 ft) and is navigable for

---

[7] "Sudokhodnye Shliuzy," by A. V. Mikhailov, Moscow, 1955.
[8] "Velikie Stalinskie Stroiki," by L. Grekulov, Moscow, 1951.

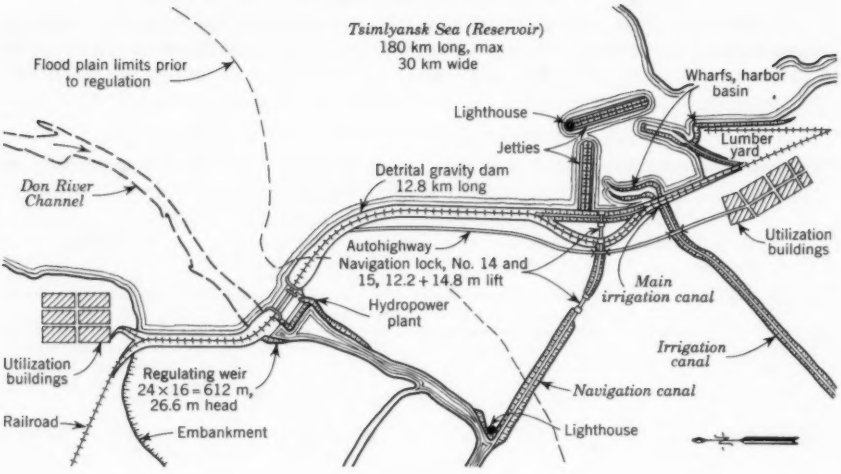


FIG. 6.—LAYOUT AT TSIMLYANSK, A TYPICAL SOVIET HYDRAULIC CENTER

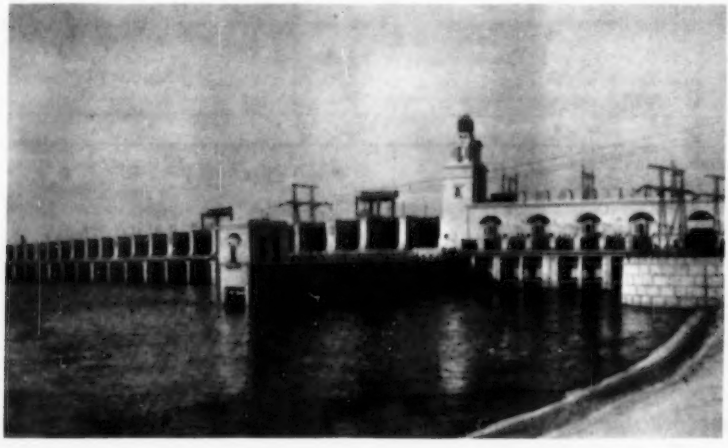


FIG. 7.—DOWNSTREAM VIEW OF THE TSIMLYANSK HYDRAULIC CENTER

3,000-ton vessels for 165 days a year. It was constructed in 20 months in 1931-32. The structures are rather primitive (partly wood); however, the reconstruction of this important strategic waterway connecting Baltic and Arctic Sea is planned, if not already under construction (as of 1960).

b. Moscow-Volga Canal, 128 km (80 mile) waterway joins the Volga at Ivankovo and the Moscow River on which large harbor facilities are developed. The canal is 5.50 m deep and uses 11 locks to overcome a 47 m (155 ft)

TABLE 1.—HYDRAULIC CENTERS OF THE GREAT VOLGA

Name of Center	Distance along Volga River, in km	Capacity Installed, in mw	Reservoir Capacity, in cu km (10^6 acre-ft)	Remarks
Volga River				
1. Ivankovo	2,810	30	-	In operation, junction Volga-Moscow Canal
2. Uglich	2,740	120	-	In operation
3. Rybinsk	2,700	330	24.8 (20)	Improvement of Volga navigation
4. Gorkiy	2,300	400	-	In operation
5. Cheboksary	1,950	800	-	Under construction
6. Kuybyshev	1,390	2,100	38.0 (31)	In operation, backwater 600 km
7. Saratov	920	1,000	-	Proposed
8. Stalingrad	480	2,100	37.0 (30)	Under construction
9. Nizhne Volzhskaya (Astrakhan)	100	-	-	Proposed
Kama River				
10. Nizhne Kamskaya	-	-	-	Proposed
11. Votinsk	-	-	-	In operation
12. Molotov-perm	-	-	-	In operation
13. Solikamsk	-	-	-	Under construction
Don River				
14. Tsimlyansk	-	160	12.8 (10.4)	In operation

elevation difference. Completed in 1941, it requires feeding of the summit pool from storage reservoirs that are a considerable distance from the canal.

c. The Marien Canal System was developed progressively in the period from 1810 to 1914 to connect the Baltic Sea and Volga River by means of several minor rivers, including Svir and Neva and Lakes Oneiga and Ladoga. The original system is 1,135 km (700 miles) long, the summit pool is located at El. 122.6 m (390 ft) above sea level. Since 1945 the system has been and the

at El. 122.6 m (390 ft) above sea level. Since 1945 the system has been modernized and the original 27 obsolete locks have been progressively replaced by 14 modern-standard structures.

Other canal junctions of the Great Volga include a complex canal and dam system of the Petchora-Vytschegda transfer south, already mentioned and under construction. The Turkmen Canal, planned to establish connection of the Caspian and Aral Seas, which was under construction in the fourth and fifth Five Year Plan, was abondoned in 1952 for reason of a basic change in

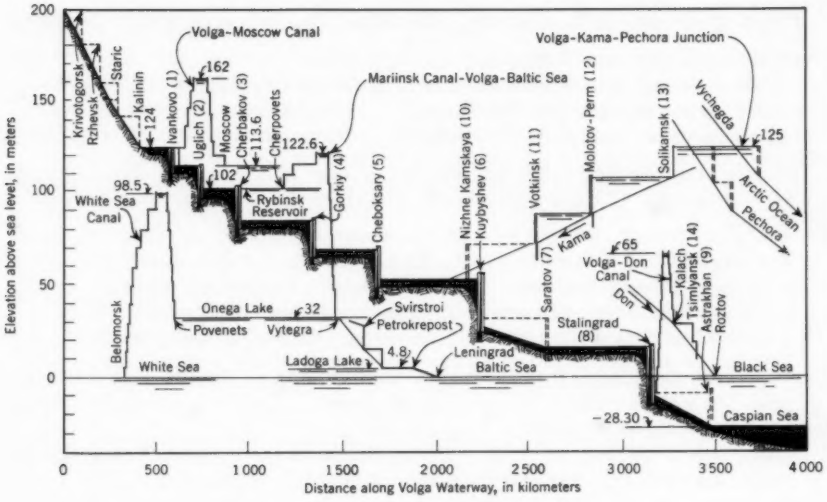


FIG. 8.—THE GREAT VOLGA WATERWAY ELEVATION SCHEME

overall hydrologic planning. Stalingrad Canal is planned together with the Stalingrad Dam and Hydraulic center. It will serve to irrigate the Pre-Caspian depression. Its construction was postponed in 1954 for the same reason as given for the Turkmen Canal.[9]

HYDROLOGY OF THE GREAT VOLGA

Hydrologic conditions prior to reconstruction were determined by a long term average annual volume of discharge of the Volga of 255 to 260 cu km (208 to 212 x 10^6 acre-ft). Of this discharge, 65% was carried in the period of melting snow (February to April). This was the season for floods, in which a discharge of 63,000 cu m per sec (2.2 x 10^6 cfs) was recorded in Saratov in 1915, creating a stage fluctuation of 17 m (56 ft). The remaining 9 months was the period of low and medium flow, when the depth was insufficient for

[9] "Les Transports Fluviaux en l'Union des R. S. S.," La Documentation Francaise, Notes and Etudes Documentaire, Paris, 1955.

navigation, even by low-draft boats. For the construction of the Kuybyshev Dam, the following hydraulic data were used:

Annual discharge:	Mean, 250 cu km (200×10^6 acre-ft)
	Maximum, 363 cu km (296×10^6 acre-ft)
	Minimum, 141 cu km (115×10^6 acre-ft)
Spring floods:	Three-month period, February to April
	Mean, 153 cu km (124×10^6 acre-ft)
	Maximum, 240 cu km (196×10^6 acre-ft)
	Minimum, 86 cu km (70×10^6 acre-ft)

Flood discharge data used for calculation of spillway and weir openings:

81,000 cu m per sec (2.85×10^6 cfs),	10,000-yr probability
71,000 cu m per sec (2.50×10^6 cfs),	1,000-yr probability
60,000 cu m per sec (2.12×10^6 cfs),	100-yr probability
52,000 cu m per sec (1.84×10^6 cfs),	20-yr probability

Riesenkampf designed the project for a flow of 260 cu km, supplemented by 44 cu km (36×10^6 acre-ft) transferred from the drainage of other rivers as follows:

Volga	260 cu km (212×10^6 acre-ft)
Oniega River Transfer	8 cu km (6.5×10^6 acre-ft)
Vytschegda River Transfer	6 cu km (4.8×10^6 acre-ft)
Don River Transfer	30 cu km (24.8×10^6 acre-ft)

Riesenkampf considered this average annual volume as the absolute minimum to meet the requirements of the original Great Volga Project of 1926, which included the stabilization of the Caspian Sea level.[10]

Hydrologic Condition in 1955.—Coinciding with the construction activities in the Volga drainage, the river's annual discharge is showing a steadily declining trend. K. K. Gjul estimates the hydrologic losses, due to the hydrotechnical constructions, to be 50 to 60 cu km annually. According to Gjul, 15,000 artificial reservoirs have been placed in operation in the Volga drainage basin up to 1953, some of them of huge proportions, as previously described. These reservoirs account for the high rate of transpiration intensity. Furthermore, the large irrigation projects, developed in the southern part of the USSR along the Volga and the change of the cultivation of the land by the Kolchoz system of farming, magnify the hydrologic losses. The application of tractor plowing, which penetrates the soil 0.40 m to 0.50 m. instead of the previously used hand plowing which reaches only 0.2 m to 0.25 m, greatly increases the consumption of water for agricultural needs, according to Gjul. Due to the changes of the Volga-Don Canal design, the 30 cu km annual transfer of the Don River discharge was abandoned and replacement was found in the Petschora and Vytschegda Rivers by means of the Kama River. This project is supposed to provide, in 1956, an additional supply of 41 cu km annually to the Volga discharge. However, calculation indicates that even then the supply of water for balancing the adverse hydrology of the Volga will be less than calculated by Riesenkampf in 1926. As a result, the decline of the

[10] "Kaspiyskoye More," by B. A. Shlyamin, Moscow, 1954.

Caspian Sea, resulting mostly from the decline of the Volga discharge, continues and is bound to continue for a considerable time, according to the Soviet scientific studies.

CONCLUSIONS

The progress of the Great Volga Waterway project for extensive territorial planning and development has been affected by many conditions during the long period of planning and construction. As the first project for hydrotechnical development of a large river basin within the Soviet economic program, the Volga reconstruction proceeded through the first three Five Year Plans to 1941. Discontinued for the duration of World War II, the project was revived in 1946 and in a revised and adjusted form continues since then. The major objective of the plan, the Volga-Black Sea junction, was achieved in 1952, and further important progress is expected in 1965, when Petchora-Kama-Volga junction will release additional water destined for the Arctic Ocean, south.

The failure of the Volga project to arrest the sinking of the Caspian Sea as originally planned and the indication of a general inadequacy of water supply needed for the efficient operation of the waterway and hydropower installations, brought up a disagreement among the leading Soviet scientists and engineers. As a result, the Soviet Academy of Sciences, Institute of Oceanography, conducted, in the period 1952 to 1957, a scientific survey of phenomena, leading to the Volga flow decline which coincides with the construction activities in its drainage. The symposium published on this occasion, in 1958, extends far beyond the limits of conventional hydrology in the effort to establish criteria for the correlation of long term hydrometeorologic cycles, Volga flow, and Caspian Sea sinking. The human interference with the natural discharge conditions of the Volga is mentioned as only one of the factors causing the Volga and Caspian Sea decline. However, higher emphasis in the document is placed on long term solar activities and climatic cycles.

Despite the technical deficiencies, the Great Volga represents an important superarterial waterway across the European part of Russia. The second largest river of Europe, the Danube, which also empties into the Black Sea, is of great importance to transportation for Central and Eastern Europe. Since the World War II the Soviets took over, together with the satellite countries, the administration of this waterway and are using extensively her transportation facilities. Specially designed transportation vessels for use in combined coastal seas-inland waterway traffic, may operate at present between Budapest and Moscow, via Danube, Black Sea, and the Great Volga.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

DISCUSSION

Note.—This paper is a part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. WW 1, February, 1961.

ENGINEERING ASPECTS OF COASTAL SEDIMENT MOVEMENT[a]

Closure by R. Silvester

R. SILVESTER.[1]—Much valuable research has been carried out on the mode of sediment movement under the influence of wave action, including the outstanding work of the discussers. Results from this research have led to a greater understanding of the forces involved and a more rational approach to beach processes. But, to the writer, it seems a little ambitious to think that the relationships obtained between wave and sediment characteristics under ideal laboratory conditions can be applied quantitatively in nature where waves are for ever changing height, period, and direction.

Even if it were possible to predict the beach profile accurately for any given particle conditions, this is but a part of the larger problem of sediment transport along the coast. It is the acceleration or deceleration of this longshore drift that determines the beach line at any point. The transient profile of this beach line is of secondary importance. A macroscopic rather than a microscopic view needs to be taken, the beach should not be studied at the expense of the Admiralty chart or map.

The discussers have touched on two of the present-day difficulties, namely sediment sampling and wave measurement. The author would specify these problems more closely as: (a) the need to measure sediment movement over long periods, say, one year; and (b) the necessity for recording wave direction over similar periods of time.

In respect of model studies, it is stated that the effect of rocky headlands and longshore currents have not been studied to date. Investigations carried out at the University of Western Australia into this aspect of coastal engineering are to be published shortly.[2] These dealt with the equilibrium shape of coastlines in plan. With fixed headlands and persistent swell from one direction, the model sedimentary coast assumed a particular shape that, in fact, minimized the longshore movement of material. Where this shape is recognizable on natural coastlines the direction of net sediment movement can be ascertained. This investigation also led to methods of minimizing or preventing longshore drift.

The writers anticipate that, as the economics of maritime operations demand it, and the development of better instruments permit it, a more comprehensive analytical solution will be produced. It is to be hoped that this will be so, but a limiting factor in the application of such theories is the dearth of engineers serving as intermediaries between the pure scientist and

---

[a] September, 1959, by R. Silvester (Proc. paper 2168).

[1] Dept. of Civ. Engrg., Univ. of Western Australia, Nedlands, W. A.

[2] "Stabilisation of Sedimenatry Coastlines," by R. Silvester, Nature, London, (in press).

the practical engineer involved in the design of ports and coastal defences. There is a need for research engineers who would have time to read and digest the output of the scientist, keep in close touch with engineering problems, and be able to fill in the gaps of knowledge by conducting some model research. There has always been a lamentable time lag between scientific discoveries and their application by the engineering profession.

The comments of the discussers are appreciated.

AN ELECTRIC ANALOG MODEL OF A TIDAL ESTUARY[a]

Closure by H. A. Einstein and James A. Harder

H. A. EINSTEIN[8] and JAMES A. HARDER.[9]—The authors agree with G. B. Fenwick that it would have been very desirable to have included some of the very interesting results that were obtained from the analog model of the Delta Region in the paper and to have indicated their engineering significance. Under an agreement with the California Department of Water Resources, the University of California agreed not to publish these results without their approval. The results of our study will be used in an evaluation of several possible developments within the Delta Region, and while these results will bear on the feasibility of some of the plans, they will not necessarily be decisive.

Members of the ASCE will appreciate that there are many interested groups of citizens, who, supported by the partial results of a feasibility study, would argue powerfully for a particular course of action (in their own economic interest) before all the factors necessary to the decision can be made available to the public.

It is true that within the channels leading to the Delta Region there is sufficiently intense mixing so that the vertical salinity gradients are small. Thus, it might be hoped that the extent of salinity intrusion could be predicted from average tidal velocities. That such predictions are not yet reliable shows that we understand the nature of the mixing process imperfectly, or do not have the computing techniques sufficiently well developed. However, even if quantitative predictions of the change of salinity distributions cannot be made today with sufficient reliability, it is possible to make qualitative predictions. It might be argued that were the upstream area subject to tidal fluctuations reduced, by cutting off some channels for example, the tidal flow in the channels and basins leading to this area would be likewise reduced, with a corresponding reduction in mixing and in salinity intrusion. If a hydraulic model study or an analog study showed that due to changes in the resonance patterns or due to other factors the mean range of the tide had been increased in the upstream area, increased mixing and increased salinity intrusions may be expected. If two such opposing effects leave the tidal flows in the changed channel system the same as they are now, one may draw the conclusion that the salinity conditions would remain the same too. This latter was the actual result for one of the proposed projects.

---

[a] September, 1959, by H. A. Einstein and James A. Harder (Proc. paper 2173).
[8] Prof. of Hydr. Engrg., Univ. of California, Berkeley, Calif.
[9] Asst. Prof. of Hydr. Engrg., Univ. of California, Berkeley, Calif.

THE SUPPLY AND LOSS OF SAND TO THE COAST[a]

Closure by J. W. Johnson

J. W. JOHNSON.—Since publication of this paper, several studies have been made that provide techniques and procedures leading to a better understanding of nearshore sediment movement. Work by Svasek and Engel[2] pertained to the use of radio-active tracers in studying sediment transport due to the combined action of waves and currents, and Russell[3] used fluorescent tracers in measuring the littoral drift of material along natural shingle beaches. Both of these methods provide valuable experimental tools for studying sediment movement under natural conditions.

Valembois[4] has investigated the problems involved in a model study of littoral transport and the agreement between model and nature. He also attempted to define the conditions of correct representation in a model of the movement of materials in suspension. To arrive at a procedure for estimating the rate of littoral transport in the Great Lakes, Bajorunas[5] derived an equation of transport in terms of wave energy. The numerical coefficients in the equation were determined from field observations.

Mr. Eagleson and Mr. Dean, in their discussion, outline a procedure for computing the rate at which sand can be delivered to a shoreline by the onshore movement of sand by wave action. The reach of the California coast discussed by the author is geologically one in which sediments are eroded from the land and then carried to the sea. Where currents are strong enough to carry these sediments offshore, the action of the waves will tend to move certain fractions back to the shore by the action discussed by Eagleson and Dean. However, there does not appear to be an appreciable source of sand offshore that might provide an unlimited and continuous supply of sand to the shoreline of California.[6] This condition is in contrast to the south shore of Long Island, New York, in which the old glacial deposits offshore appear to provide a continual supply of sediment to Long Island. The possible onshore movement of sand appears to be the only reasonable explanation for the 300,000 cu yd of sand per year that has been estimated to be moving as littoral

---

[a] September 1959, by J. W. Johnson (Proc. paper 2177).

[2] "Use of a radio-active tracer for the measurement of sediment transport in the Netherlands," by J. N. Svasek and H. Engel, Proceedings, 7th Conference on Coastal Engrg., 1961.

[3] "The use of fluorescent tracers for the measurement of littoral drift," by R. C. H. Russell, Proceedings, 7th Conference on Coastal Engrg., 1961.

[4] "Etude sur modele du transport littoral. Conditions de similitude," by J. Valembois, Proceedings, 7th Conference on Coastal Engrg., 1961.

[5] "Littoral transport in the Great Lakes," by L. Bajorunas, Proceedings, 7th Conference on Coastal Engrg., 1961.

[6] "Movement of sand around Southern California promontories," by Parker D. Trask, Beach Erosion Bd. Tech. Memo. No. 76, Corps of Engrs., Washington, D. C., 1955.

drift westward along the south shore of Long Island,[7] because erosion of the eastern portion of Long Island seems to be insufficient to account for this annual rate of movement.

It would be of interest to apply the principles outlined by Eagleson and Dean to the Long Island problem for a determination of the general order of magnitude of the annual contribution that might be anticipated from the offshore source.

7 "Atlantic Coast of Long Island, New York, Fire Island Inlet and Shore Westerly to Jones Beach," Beach Erosion Control Report on Cooperative Study, New York District, Corps of Engrs., (unpublished), pp. A-16, 1955.

NATURAL BY-PASSING OF SAND AT COASTAL INLETS[a]

Closure by P. Bruun, F. ASCE, and F. Gerritsen

P. BRUUN,[1] F. ASCE, and F. GERRITSEN.[2]—The authors appreciate Mr. Bowman's supplementary remarks to their statements about the Oregon Inlet in North Carolina. It is true that the inlet depected on English maps from the 16th Century (located where today's Oregon Inlet is) does not carry the name Oregon, and an inlet in this area will probably always have a constant struggle for existence. Our tide figures are for spring tides as indicated in Table 1 and, according to the United States Coast and Geodetic Survey, Department of Commerce, (USC & GS) the spring tides measure 4 ft to 5 ft on the open seashore, to which we were referring. This apparently was not made clear enough.

We share Mr. Bowman's interest in Oregon Inlet and have worked out some data that may be of interest.

Current velocities in Oregon Inlet were measured[3] in 1931 and 1932. From these measurements the volumes of the tidal prisms for flood and ebb were computed. The results are indicated in Tables 2 and 3.

It is apparent that winds have a strong influence on the quantity of flow through the inlet. Computation of the cross sections of Oregon Inlet compared with information of earlier date gives the figures indicated in Table 3.

The fact that the January, 1959 figure is somewhat smaller than that of June, 1958 is in agreement with the normal action of such an inlet. In the wintertime, littoral drift transport along the coast is generally heavier than in summer, so that more sand is entering the gorge of the inlet from the ocean tending to decrease the inlet's cross section.

The cross-section stability of the inlet can be evaluated[4] by considering the $\frac{\Omega}{M}$ (tidal prism over annual littoral drift) and $\frac{Q_{max}}{M}$ ratio $= \frac{1}{r}$. With Ω = about 80×10^6 cu yd per half cycle we find $\frac{\Omega}{M} \sim 80$ and $\frac{Q_{max}}{M} \sim 5 \times 10^{-3}$ (1). Studies of numerous inlets have shown that those with a $\frac{\Omega}{M}$ ratio < 100 have a predominant transfer of sand on bars or shoals across the inlet entrance and comparatively small tidal flow; for this reason they may be rather unstable and are usually characterized by one or more narrow, frequently shifting, channels with rather high velocities. Oregon Inlet seems to be no exception in this respect.

---

[a] December 1959, by P. Bruun and F. Gerritsen (Proc. Paper 2301).

[1] Head, Coastal Engr. Labs., Univ. of Florida, Gainesville, Fla.

[2] Assoc. Prof., Coastal Eng. Lab. Univ. of Florida, Gainesville, Fla.

[3] House Document No. 310, 81st Congress, 1st session.

[4] "Stability of Coastal Inlets," by P. Bruun and F. Gerritsen, North Holland Publishing Co., Amsterdam, 1960.

In regard to the $\frac{Q_{max}}{M}$ ratio $\sim 5 \times 10^{-3}$ it has in a similar way been found that ratios $< 10 \times 10^{-3}$ represent a less stable situation and Oregon Inlet fits well into this picture also. Although Table 3 tends to indicate a rather stable cross-sectional area within the limited time period considered, it is quite evident that Oregon Inlet, with its high $r = \frac{M}{Q_{max}}$ ratio (215), its low $\frac{Q}{M}$ ratio (80) and low $\frac{Q_{max}}{M}$ ratio $\left(5 \times 10^{-3}\right)$, is not a very good navigation inlet.

With reference to remarks by Mr. Bowman this may explain why dredging has not been very effective. If a major lasting improvement is to be obtained, dredging will probably have to be carried out all the way through from deep

TABLE 2.—FLOW CHARACTERISTICS OF OREGON INLET

Date	Wind from	Total flow on flood tide tidal prism	Max. inst. flood flow, cu ft per sec.	Total flow on ebb tide, acre feet	Max. inst. ebb flow, cu ft per sec	Max. velocities	
						ebb fps.	flood fps.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Sept. 9, '31	N.E.	47,769	134,050	37,399	89,150	2.3	2.5
Aug. 31, '32	W.	42,726	129,100	40,054	102,700	2.6	2.4
Oct. 11, '32	S.W.	34,873	126,500	57,208	127,300	3.2	2.4

TABLE 3.—GORGE AREAS OF OREGON INLET

Date	Cross section MLW, in square feet	Cross section MHW, in square feet	Source of Information
(1)	(2)	(3)	(4)
1931-32	39,400	53,000	H. D. 310 81st Congr., 1st session
June 1958	43,300	54,400	C. of E.[a] maps
Jan. 1959	40,000	51,000	C. of E[a] maps

[a] United States Army, Corps of Engrs.

ocean to deep bay. An improvement could, theoretically, be obtained by increasing Q_{max}, but a desirable increase to about twice the present value would cause considerable flooding in the bay area during storm tide conditions. The improvement would probably not be very stable unless it was accompanied by extensive jetty construction, that, in turn, would cause a serious leeside erosion problem on the downdrift or south side.

We appreciate Mr. Bowman's remarks very much and hope to have clarified the matter with the preceding details.

BREAKWATERS IN THE HAWAIIAN ISLANDS[a]

Discussion by Charles E. Lee and Robert L. Miller

CHARLES E. LEE,[7] F. ASCE.—By discussing design and construction experiences in Hawaii, Mr. Palmer has indeed demonstrated that breakwater design has developed much beyond the old concept of "build it as the one at the other location." This intuitive school of design, that is the method of comparison of characteristics of phenomenon at a proposed harbor site with another, without actual determination of characteristics at either site, has resulted in overdesign at some locations and excessive maintenance at others. Each site has its individual characteristics and structures must be designed accordingly. It is agreed that much has been learned concerning proper breakwater and jetty design, particularly since 1950. However, there also exists the problem of promoting greater acceptance and use of these recent developments in design methods by those responsible for design. The importance of experience must not be discounted, for this is an irreplaceable factor in determination of characteristics of the site and the reasonable and practical application of design methods.

The brief comments on the masonry or laid-up construction of the Port Allen breakwater are heartily endorsed. However, in areas free of surf, stone can be placed below the water surface with the aid of divers. This method is used at some locations in the Great Lakes. Data are not available to permit application of the WES stone size guide formula[8] to this placement method because of the high friction factor involved. The WES formula is based on pell mell placement of stone with the result that there is less keying action than obtained in conventional stone placement methods, but yet with greater keying than obtained in the prototype by end-dump placement. It may be considered that for conventional placement, or for laid-up placement, the friction factor is incidently included as a safety factor in the K_Δ constant along with the effects of other neglected variables such as shape factor, possible scale effects, continuity of wave attack, and so on. Therefore, the formula might be modified to be applicable to masonry type structures by determination of the proper K_Δ constant. However, even if the designer feels that a safe structure has been designed, he may still have a doubt as to the accuracy with which the stone below water surface can be placed during construction. In addition to high placement cost for masonry construction, consideration must be given to the added cost of special quarrying methods and blasting techniques to obtain the required regularity of shape of the stone. It appears then

[a] June 1960, by Robert Q. Palmer (Proc. paper 2507).

[7] Asst. Chf., Hydr. Design Branch, Engrg. Div., Civ. Works, Office, Chf. of Engrs., Washington, D. C.

[8] "Laboratory Investigation of Rubble-Mound Breakwaters," by Robert Y. Hudson, Proceedings, ASCE, September, 1959, p. 93.

that this type of design may be used only when reasonably long periods of calm may be expected, and when the lack of an economical source of sufficiently large stone justifies the added cost of placement and quarrying.

The author stated that in model tests of Nawiliwili Harbor stones were moved upslope to the crest of the structure. The results of such action have been noted at several locations in nature. In June, 1960 a stone of about 7 tons to 8 tons in weight and completely covered by barnacles (indicating prior emersion) was noted on the crest of the main breakwater at Crescent City, California. Stones were also noted beyond the leeward toe of the structure at which there was no apparent damage to the leeward slope. In this same general section stones had also been moved downslope and beyond the seaward toe. Engineers familiar with the site indicated that both types of movement probably occurred during the storms of February, 1960. This is considered to illustrate the variability of wave characteristics and energy distribution occurring at any one site and incidently applicable to a single storm.

Several of the questions proposed in the conclusions' section are currently under investigation through hydraulic model studies. These studies are being performed under the Civil Works Investigation Program of the Chief of Engineers, the program that sponsored the studies that enabled Mr. Hudson to develop the WES stone size guide formula. This test series, CW-815, "Stability of Rubble-Mound Breakwaters," is being conducted at the U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., and is now being directed toward the design of breakwater heads.

The results of the project study conducted for Nawiliwili Harbor are being integrated into this study. It is hoped to initially determine the hydraulic phenomenon involved and to devise methods to overcome the problems. Among the features to be tested are the high velocity jet, and of broken, breaking, and unbroken waves by varying H, T, and d (where d is the still water depth in feet), thereby giving consideration to various ranges of d/L, H/L, and H/d. This should give information on the variation in degree of damage to the structure resulting from variation in distance between the point of wave breaking and the structure.

Another study of interest in breakwater design is CW-847, "Scale Effect Tests of Rubble Breakwaters." This series of tests are being performed in the Beach Erosion Board 6-ft wave tank by personnel of the staff of that Board in cooperation with personnel of the Waterways Experiment Station. Tests have been made using stone components and further tests are scheduled for fiscal year 1961 using quadrapods, a concrete component with four legs.[9] The acquisition of this data will provide for possible improvement of the WES formula for size of components by indicating if there is a scale effect involved. The information on quadrapods will be especially helpful because the use of a formed shape of component eliminates the shape variable and reduces a possible placement variable. There is also no available results to report on this study at this time (1960).

Other federal agency investigations of interest to breakwater design, proposed or underway, include studies of wave force, sand bypassing and structure absorbing qualities. Many educational institutions have programs of basic or applied research that have in the past and will, in the future, be of great aid. However, all concerned with harbor design cannot emphasize too

9 "Design of Quadrapod Cover Layers for Rubble-Mound Breakwaters," by R. A. Jackson, Misc. Paper No. 2-372, U. S. Army Waterways Experiment Station, January 1960.

much, to both educational institutions and governmental agencies, the need of continued accelerated research and investigation of oceanographic prcblems. The investigations should not overlook the necessity of prototype observation. Much can be learned from detailed analysis of existing structures and the forces acting thereon.

ROBERT L. MILLER.[10]—One of the most vexing questions that a marine geologist has to contend with is, "what are the conditions required for stability in a natural shoreline?" One fairly well documented point by Zeigler, Hayes and Tuttle[11] is applicable to this question. Over a period of time of the order of a decade, major storms will cause considerable local change on sand beaches but on a "stable" coast, the original configuration may come back in a matter of weeks. However, to the best of the writer's knowledge there has been little done in investigating the fundamental mechanics of erosion on rocky coasts, from the quantitative point of view. It appears that the marine geologist may profitably study papers on the construction and history of breakwaters as an analogue to the problem previously stated. An excellent example is the author's paper.

The writer would like to raise several questions that reflect his own interests.

1. Consider the case in which a shoal, flat, area projects for some distance seaward of the breakwater as at Kawaihae harbor. Is there a possibility that for some critical value for local wave height, H, the breaking wave will reform at a value $H_1 < H$ and break again on or close to the structure? The implication is that for a wave large enough, the second breaker may still have considerable force with which to attack the structure, despite energy dissipation over the intervening rough coral surface between the first breaker and the breakwater. This is particularly a possibility during a conjunction of storm tides and storm waves that approach from a frontal direction.

2. The reflection characteristics of two types of sloping breakwaters examined by Palmer are of particular interest. It appears to me that conditions for both erosion and deposition on the seaward side may vary markedly between a structure with an armor layer of large fitted and keyed stones as at Port Allen, and one using tribars, as at Nawiliwili harbor.

In this connection, Lewis[12] has pointed out that the backwash is very much stronger than the swash on shingle beaches under destructive wave action. As a result, shingle is rapidly removed and carried seaward to deeper water during storm periods. The inference may be made in the absence of other evidence, that backwash is strongest by far on a tribar (or tetrapod) armored structure, and the swash is strongest on the relatively smooth fitted stone armored structure.

If this is the case, one may expect that sedimentation would occur more rapidly and to a greater extent adjacent to smooth breakwaters, than adjacent to those with very rough surfaces, that is composed of tribars. This phe-

---

[10] Assoc. Prof., Dept. of Geology, Univ. of Chicago, and Assoc. in Marine Geology, Woods Hole Oceanographic Inst.

[11] "Beach Changes During Storms on Outer Cape Cod, Massachusetts," by J. M. Zeigler, C. R. Hayes, and S. Tuttle, Journal of Geology, Vol. 67, No. 3, 1959, pp. 318-336.

[12] "The Effect of Wave Incidence on the Configuration of a Shingle Beach," by W. V. Lewis, Journal of Geology, Vol. 78, 1931, pp. 129-148.

nomenon would be accentuated by the fact that for other reasons, tribar armored structures may be built with more steeply sloping outer walls. An immediate effect of extensive seaward silting would be to decrease the still water level with time, thus invalidating the original design wave computations, and increasing the liklihood of over-topping.

Whether such an apparent advantage of tribar armoring is real or not will actually depend on observations over a long period of time. The complexity of forces, and the individual variation from one breakwater location to the next make it likely that no other method of prediction is presently (1960) available.

TETRAPODS AND OTHER PRECAST BLOCKS FOR BREAKWATERS[a]

Discussion by Shoshichiro Nagai and A. S. Kahan

SHOSHICHIRO NAGAI.[4]—The author emphasized the following four points as the most essential factors for a precast concrete block used as the cover layer of a breakwater: (a) to satisfy the hydraulic condition of dissipating and absorbing wave energy, (b) to be sufficiently strong to meet the conditions in which it is to be used, (c) to be easy to manufacture and place in site, and (d) to be reasonably priced. The writer agrees quite with him on that opinion. It is true that the tetrapod has the four factors, and it is one of the most prominent precast blocks in use (1960) as the cover layers of breakwaters, but it must also be recognized that the legs of the tetrapod are comparatively easily broken by dropping and colliding with each other, as proved by experiences in site and prototype tests. The cost may not always be cheap compared with other precast blocks, probably because of the high royalty.

The following conclusions are noted from the results of a number of laboratory tests that have been completed in an open wave channel, that is 25 m long, 2 m wide, and 1 m deep as well as in a closed wave channel with a wind blower, that is 23 m long, 1 m wide, and 1 m deep.

1) Two layers composed of the concrete hollow tetrahedron armor units have better characteristics for absorbing wave energy or wave pressure on breakwaters and for stability against breaking waves than the other specially shaped concrete armor units, such as tetrapods, and so on. When the two layers of the hollow tetrahedron armor units are used for protective cover layers on the seaward slopes of the rubble-mounds of composite-type breakwaters, it will be expected to obtain approximately 30% to 50% greater attenuation of shock pressures exerted by breaking waves on the vertical walls than that due to tetrapod armor units.

2) When the two layers of the hollow tetrahedron armor units are used for the protective cover layers of parallel dykes located in front of sea-walls, the attenuation of wave run-up on the slopes of the sea-walls will be obtained to a great extent, and they will be materially effective to prevent overtopping from the sea-walls.

3) The hollowness of a hollow tetrahedron block was proved optimum in the case of a porosity of 25%, from the viewpoints of wave energy absorbing ability as well as of the strength of a block.

4) The number of the concrete hollow tetrahedron blocks necessary for two-layer placing on a given area is on an average 15% fewer than that of tetrapod blocks with the same dead weight.

---

[a] September, 1960, by P. Danel, E. Chapus, and R. Dhaille (Proc. paper 2590).

[4] Prof. of River and Harbor Engrg., Faculty of Engrg., Osaka City Univ., Osaka, Japan.

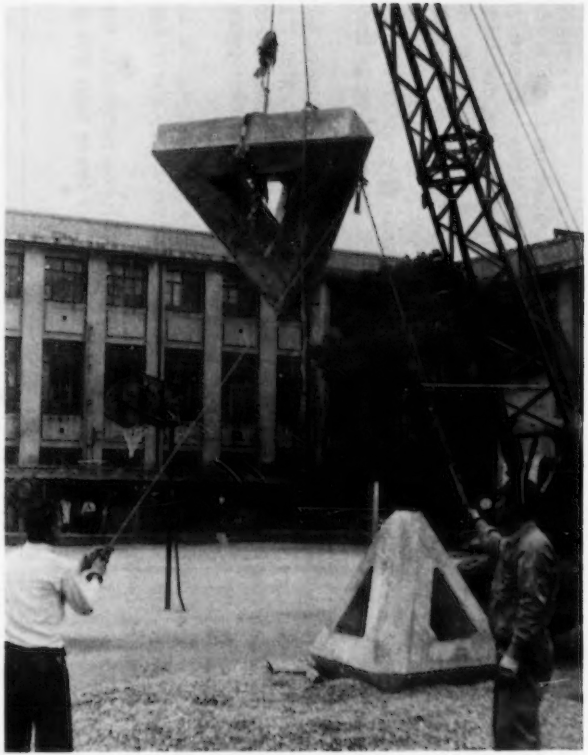


FIG. 12.—FALL FROM 3,5 m. HEIGHT ON A GRAVEL LAYER

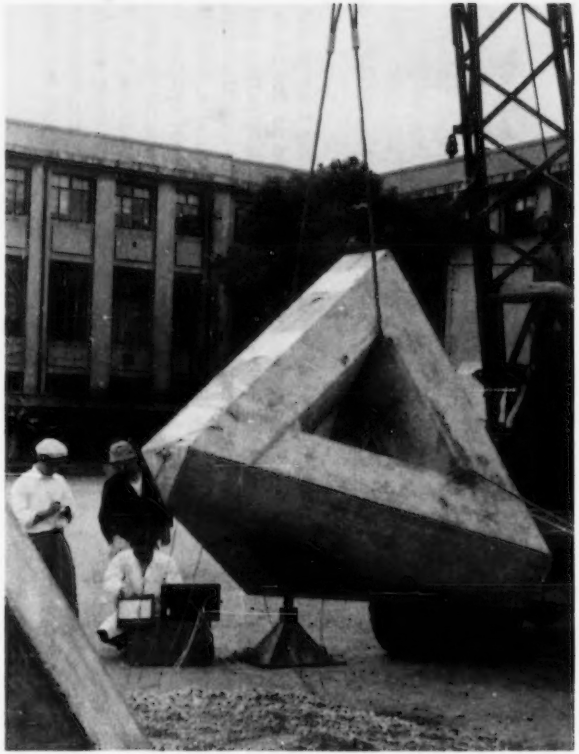


FIG. 13.—RECORD OF EDGE STRESS DURING SUSPENSION

The mechanical strength of the hollow tetrahedron block was specially considered from the practical aspects of use in prototype conditions. The field tests of prototype 2-ton concrete, hollow, tetrahedron blocks with a

FIG. 14.—TEST OF EDGE STRESS DURING SUSPENSION

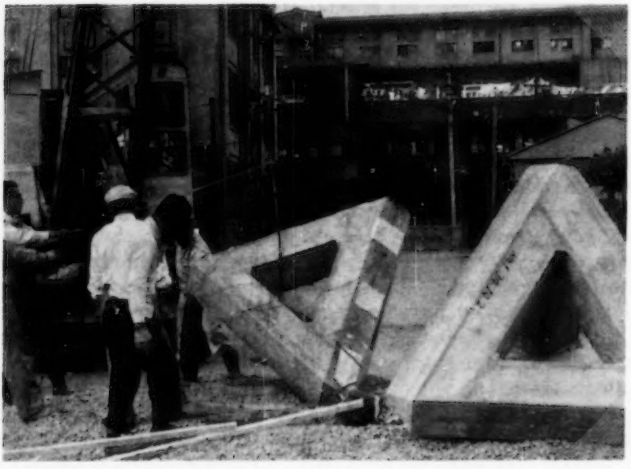


FIG. 15.—MUTUAL COLLIDING

porosity of 25% (without any reinforcement) was performed to find the strength against very roughly dumping in seas by the measures of falling down on gravel layers and on hollow tetrahedron blocks from some 3.5 m height as shown in Figs. 12, 13, 14, and 15.

From the tests, the concrete hollow tetrahedron block was confirmed as being very tough against rough piling up or hard mutual colliding because of the action of resisting as one frame-work. As is known, the lifting and placing of the hollow tetrahedron block in seas are easily same as the tetrapod.

The details of the hydraulic model tests in the open and the closed wave channels have been published.[5]

A. S. KAHAN,[6] M. ASCE.—Taking into consideration that most estuaries, bays, and gulfs in civilized areas were easily utilized to serve as natural or partially natural harbors, governments and authorities are now concerned mainly in reconstruction and enlarging these existing ports or building new deep-sea ports on exposed coasts. The use of the various simple but patented precast concrete blocks instead of hardly available large natural rock blocks is frequently the only possible answer that the owner or the designer may have.

The authors gave a useful list and description of various aspects that have to be considered while designing and applying the various artificial concrete blocks and especially the tetrapods as an armour layer against wave action. To this list of considerations a few additional ones will be mentioned herein.

Curved alignments.—Most breakwaters constructed recently pertrude from the unprotected coastline to the deep-water contours, not only oblique to the prevailing wave fronts and depth-contours, but also they follow a general curved alignment so as to enclose maximum areas of protected basin with the minimum length of structure. These curvatures bring forward serious problems in the design due to the tangential forces acting on the artificial concrete blocks. Most of these blocks are sensitive to these forces, and three dimensional wave-tank tests prove the necessity of using blocks of much larger size then otherwise required for frontal attack reproduced in flume tests. In this respect the natural stone blocks behave much better, having in the tangential direction a better interlocking effect and less projective surface than the artificial blocks.

Round-heads.—Another important problem is the protection of breakwater and jetty ends or round-heads. These ends are impounded, in addition to the most serious wave heights, by tangential turbulant action of the reflected sweeping waves. Many times artificial blocks, as large as double in size, have to be used compared to frontal attack requirements or as an alternative solution armour layer slopes are to be multiplied two or three times.

Core material.—Very little has been said or tested in connection with the requirements for changing secondary armour or core covering layers when using artificial blocks as exposed armour layer. The most important characteristic of these blocks is the saving in material through increasing the porosity of the overall layers by choosing the most suitable mechanical strength and hydraulic properties. When increasing this porosity it is true that wave force is countercepted by the turbulent flow in the cavities; at the same time a large quantity of active water mass comes into contact with secondary and other core layers that very often require increase in stone-class size; they especially require the adding of filler content against the tru-flow of water masses from the sea-side to the harbor-side of the protective breakwater causing increased and undesired agitation in the harbour basin and slips.

---

[5] Proceedings, Seventh Conf. on Coastal Engrg., August, 1960.
[6] Harbors and Coasts Engr., Ministry of Transport and Communication, Israel.

Maintenance.—Though it was not fully proved, experience varified that maintenance repair of tetrapod layers is much easier than those of rectangular concrete block layers. But model-tests also proved that those tetrapods, inserted into the structure as a repair measure against eroded tetrapods, were the same ones that were repeatedly washed out by reoccurance of similar or greater storms.

Economic considerations.—While considering the saving or increase of the structure's cost, the two most important considerations are whether the problem concerns a repair job or is it a new structure, and whether small or large quantities of the artificial armour units are required. The patent-fees asked for the use of the artificial armour blocks, vary greatly with the total scope of the work. On the other hand the contractor's price depends on whether, for the casting, curing and placing large quantities are involved. Various papers[7,8] by R. Iribarren and C. Nogales have brought before coastal engineers many examples for the varification of their well known formulas. These examples, not only from reduced scale model tests but also from full scale existing rubble-mound breakwaters did show the behavior of the natural armour stone block at wave action. R. Q. Palmer[9] also gave a very interesting description on the practical use of the new invention, "the tribar" in Hawaiian breakwaters and its behaviour during gales. On the other hand the authors, beside the paper under discussion, Mr. Danels's 1954 paper,[10] and some descriptive notes in engineering journals, did not give us any detailed data on the results of the actually constructed tetrapod armour layers under various hydrographical and oceanographical conditions at some 50 or more places in all parts of the world. The breakwater designer has to use, either the stability curves given by Mr. Danel,[10] that the authors now themslf state to be in many cases obsolete and not applicable, or use formulas and coefficients found by model tests in the Waterways Experiment Station of the U. S. Corps of Engineers by R. Y. Hudson and others.[11,12] Though the only answer in any case is the reduced scale model test varification, still the safety factor so used, due to the unknown and unpublished results of the behavior of tetrapods in actual structures, causes unnecessarily increased costs in budgeting and in paid expenditure.

It would be useful if engineers all over the world would pool field information on the recently designed and built armour layers of the various natural blocks and patented artificial blocks and use the information of control profile measurements of unit movements under known wave and sea conditions. This practice will by all means advance the knowledge in this field of coastal engineering.

It may be of interest to some engineers, to quote the comparative tabulation made for the use of natural blocks or tetrapod armour,[13] taking into consid-

---

[7] R. Iribarren and C. Nogales, Revista de Orbas Publicas, Madrid, 1950.

[8] R. Iribarren and C. Nogales, Bulletin No. 39-P.I.A.N.C., 1954.

[9] "Breakwaters in the Hawaiian Islands," by Robert Q. Palmer, Proceedings, ASCE, Vol. 86, No. 2, June, 1960.

[10] P. Danel, Proceedings, Fourth Coastal Engrg. Conference, 1954.

[11] R. Y. Hudson and R. A. Jackson, Technical Memorandum, No. 2-365, WES, 1953.

[12] "Laboratory Investigation of Rubble-Mound Breakwater," by R. Y. Hudson, ASCE, Vol. 85, No. 3, September, 1959.

[13] "Tetrapod, protection against waves," by A. S. Kahan, Journal, Assn. of Engrs. and Archts. in Israel, Vol. XVII, Nos. 11, 12, November, December, 1959. In Hebrew, French translation by M. Rosenblum, Grenoble, France.

TABLE 1.—COMPARATIVE ECONOMIC TABULATION

Design Values		Wave and H = 4.00 m d = 5.00 m t = 8 sec H/l = 0.04	
(1)		Rock, 10 tons (2)	Tetrapod, 5 tons (3)
Harbour side slope, i		1:1 1/2	1:1 1/2
Sea side slope, i		1:1 3/4	1:1 1/3
Wave Run-up factor, R/H		1.12	1.15
Crown-wall top level above M. S. L.		+4.50	+4.50
Natural rock volume for B. W. section, cu m per m. r.	Gross	149.50	114.70
	Net	89.70	68.80
Thickness of double layer of tetrapods, in meters		...	3.00
Covering surface of tetrapods, sq m per m. r.		...	16.00
Tetrapod units per 100 sq m		...	70
Total tetrapod units, nos. per m. r.		...	11
Total tetrapod concrete, ton per m. r.		...	55
Total rock, ton per m. r.		236	182
Total crown-wall concrete, cu m per m. r.		6.75	6.75
Total cost of rock I. L./M.R.-B.W.		2950.0	2250.0
Total cost of tetrapod, I. L./M.R.-B.W.		...	1898.0
Total cost of crown-wall, I. L./M.R.-B.W.		459.0	459.0
General B.W. cost, I. L./M.R.		3,409.0	4,607.0
Saving, in%		26	...

eration east-mediterranean conditions and comparing design data and results for three various structure crossections: one for a fishing port or marinas, one of a medium grade commercial port, and one for a modern deep-draft commercial port. The following basic data has been used while preparing Table 1:

1. Iribarren's stability formula as revised by Hudson.[14]
2. All units in metric system.
3. Maximum available natural rock block : 10 tons.
4. γ_f specific weight of sea water : 1.03 ton per cu m.
5. γ_r specific weight of natural rock : 2.63 ton per cu m.

14 "Wave Forces on Breakwaters," by Robert Y. Hudson, Transactions, ASCE, Vol. 118, 1953, p. 653.

—ALTERNATIVE BREAKWATER DESIGN

Armour Characteristics

H = 6.00 m d = 10.00 m	t = 10 sec H/l = 0.0385	H = 8.00 m d = 15.00 m	t = 12 sec H/l = 0.0335
Rock, 10 tons (4)	Tetrapod, 16 tons (5)	Rock, 10 tons (6)	Tetrapod, 40 tons (7)
1:1 1/2	1:1 1/2	1:1 1/2	1:1 1/2
down to -6m : 1:4 1/2 up to -6m : 1:1 1/2	down to -6m : 1:1 1/3 up to -6m : 1:1 1/2	down to -8m : 1:8 1/2 up to -8m : 1:1 1/2	down to -8m : 1:11/3 up to -8m : 1:1 1/2
0.80	1.15	0.65	1.15
+5.00	+7.00	+6.00	+9.50
692	428	2049	919
415	257	1229	551
. . .	4.00	. . .	5.20
. . .	23.00	. . .	32.00
. . .	30	. . .	16
. . .	7	. . .	6
. . .	112	. . .	240
1095	675	3240	1445
8.25	21.00	8.25	41.50
13,688.0	8,438.0	40,500.0	18,063.0
. . .	3,864.0	. . .	8,280.0
561.0	1,428.0	561.0	2,822.0
14,249.0	13,730.0	41,061.0	29,165.0
. . .	4	. . .	29

6. γ_t specific weight of concrete for tetrapods : 2.40 ton per cu m.
7. μ_r friction coefficient of natural rock blocks : 1.05.
8. μ_t friction coefficient of tetrapod armour units : 1.10.
9. Porosity of natural rock block armour layer : 40%.
10. Porosity of natural tetrapod armour layer : 50%.
11. Cost of natural rock (average for all classes), in place : 12.50 I.L. per ton (1 Dollar equals 1.80 I.L.).
12. Cost of tetrapod, in place : 82.50 I.L. per cu m.
13. Cost of crown wall concrete, cast in situ : 68.-I.L. per cu m.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For title and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2703 is identified as 2703(ST1) which indicates that the paper is contained in the first issue of the Journal of the Structural Division during 1961.

VOLUME 86 (1960)

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(ST2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SM1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(ST2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(ST2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SM1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SM1), 2389(ST2)[c], 2391(SM1)[c], 2391(PO1)[c].

MARCH: 2392(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(SM1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST2), 2411(SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY2), 2416(HW1), 2417(HW3), 2418(HW1)[c], 2419(WW1)[c], 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)[c], 2426(HY2)[c], 2427(ST2)[c].

APRIL: 2428(ST4), 2429(HY4), 2430(PO2), 2431(SM2), 2432(PO2), 2433(ST4), 2434(EM2), 2435(PO2), 2436(ST4), 2437(ST4), 2438(HY4), 2439(EM2), 2440(EM2), 2441(ST4), 2442(SM2), 2443(HY4), 2444(ST4), 2445(EM2), 2446(ST4), 2447(EM2), 2448(SM2), 2449(HY4), 2450(ST4), 2451(HY4), 2452(HY4), 2453(EM2), 2454(EM2), 2455(EM2)[c], 2456(HY4)[c], 2457(PO2)[c], 2458(ST4)[c], 2459(SM2)[c].

MAY: 2460(AT1), 2461(ST5), 2462(AT1), 2463(AT1), 2464(CP1), 2465(CP1), 2466(AT1), 2467(AT1), 2468(SA2), 2469(HY5), 2470(ST5), 2471(SA3), 2472(SA3), 2473(ST5), 2474(SA2), 2475(ST5), 2476(SA3), 2477(ST5), 2478(HY5), 2479(SA3), 2480(ST5), 2481(SA3), 2482(CO2), 2483(CO2), 2484(HY5), 2485(HY5), 2486(AT1)[c], 2487(CP1)[c], 2488(CO2)[c], 2489(HY5)[c], 2490(SA3)[c], 2491(ST5)[c], 2492(CP1), 2493(CO2).

JUNE: 2494(IR2), 2495(IR2), 2496(ST6), 2497(EM3), 2498(EM3), 2499(EM3), 2500(EM3), 2501(EM3), 2502(EM3), 2503(PO3), 2504(WW2), 2505(EM3), 2506(HY6), 2507(WW2), 2508(PO3), 2509(ST6), 2510(EM3), 2511(EM3), 2512(ST6), 2513(HW2), 2514(HY6), 2515(PO3), 2516(EM3), 2517(WW2), 2518(WW2), 2519(EM3), 2520(PO3), 2521(HY6), 2522(SM3), 2523(ST6), 2524(HY6), 2525(HY6), 2526(HY6), 2527(IR2), 2528(ST6), 2529(HW2), 2530(IR2), 2531(HY6), 2532(EM3)[c], 2533(HW2)[c], 2534(WW2), 2535(HY6)[c], 2536(IR2)[c], 2537(PO3)[c], 2538(SM3)[c], 2539(ST6)[c], 2540(WW2)[c].

JULY: 2541(ST7), 2542(ST7), 2543(SA4), 2544(ST7), 2545(ST7), 2546(HY7), 2547(ST7), 2548(SU2), 2549(SA4), 2550(SU2), 2551(HY7), 2552(ST7), 2553(SU2), 2554(SA4), 2555(ST7), 2556(SA4), 2557(SA4), 2558(SA4), 2559(ST7), 2560(SU2)[c], 2561(SA4)[c], 2562(HY7)[c], 2563(ST7)[c].

AUGUST: 2564(SM4), 2565(EM4), 2566(ST8), 2567(EM4), 2568(PO4), 2569(PO4), 2570(HY8), 2571(EM4), 2572(EM4), 2573(EM4), 2574(SM4), 2575(EM4), 2576(EM4), 2577(HY8), 2578(EM4), 2579(PO4), 2580(EM4), 2581(ST8), 2582(ST8), 2583(EM4)[c], 2584(PO4)[c], 2585(ST8)[c], 2586(SM4)[c], 2587(HY8)[c].

SEPTEMBER: 2588(IR3), 2589(IR3), 2590(WW3), 2591(IR3), 2592(HW3), 2593(IR3), 2594(IR3), 2595(IR3), 2596(HW3), 2597(WW3), 2598(IR3), 2599(WW3), 2600(WW3), 2601(WW3), 2602(WW3), 2603(WW3), 2604(HW3), 2605(SA5), 2606(WW3), 2607(SA5), 2608(ST9), 2609(SA5)[c], 2610(IR3), 2611(WW3)[c], 2612(ST9)[c], 2613(IR3)[c], 2614(HW3)[c].

OCTOBER: 2615(EM5), 2616(EM5), 2617(ST10), 2618(SM5), 2619(EM5), 2620(EM5), 2621(ST10), 2622(EM5), 2623(SM5), 2624(EM5), 2625(SM5), 2626(SM5), 2627(EM5), 2628(EM5), 2629(ST10), 2630(ST10), 2631(PO5)[c], 2632(EM5)[c], 2633(ST10), 2634(ST10), 2635(ST10)[c], 2636(SM5)[c].

NOVEMBER: 2637(ST11), 2638(ST11), 2639(CO3), 2640(ST11), 2641(SA6), 2642(WW4), 2643(ST11), 2644(HY9), 2645(ST11), 2646(HY9), 2647(WW4), 2648(WW4), 2649(WW4), 2650(ST11), 2651(CO3), 2652(HY9), 2653(HY9), 2654(ST11), 2655(HY9), 2656(HY9), 2657(SA6), 2658(WW4), 2659(WW4)[c], 2660(SA6), 2661(CO3), 2662(CO3), 2663(SA6), 2664(CO3)[c], 2665(HY9)[c], 2666(SA6)[c], 2667(ST11)[c].

DECEMBER: 2668(ST12), 2669(IR4), 2670(SM6), 2671(IR4), 2672(IR4), 2673(IR4), 2674(ST12), 2675(EM6), 2676(IR4), 2677(HW4), 2678(ST12), 2679(EM6), 2680(ST14), 2681(SM6), 2682(IR4), 2683(SM6), 2684(PP5), 2685(IR4), 2686(EM6), 2687(EM6), 2688(EM6), 2689(EM6), 2690(EM6), 2691(EM6)[c], 2692(ST12), 2693(ST12), 2694(HW4)[c], 2695(IR4)[c], 2696(SM6)[c], 2697(ST12)[c].

VOLUME 87 (1961)

JANUARY: 2698(PP1), 2699(PP1), 2700(HY1), 2701(SA1), 2702(SU1), 2703(ST1), 2704(ST1), 2705(SU1), 2706(HY1), 2707(HY1), 2708(HY1), 2709(PO1), 2710(HY1), 2711(HY1), 2712(ST1), 2713(HY1), 2714(PO1), 2715(ST1), 2716(HY1), 2717(SA1), 2718(SA1), 2719(SU1)[c], 2720(SA1)[c], 2721(ST1), 2722(PP1)[c], 2723(PO1)[c], 2724(HY1)[c], 2725(ST1)[c].

FEBRUARY: 2726(WW1), 2727(EM1), 2728(EM1), 2729(WW1), 2730(WW1), 2731(EM1), 2732(SM1), 2733(WW1), 2734(SM1), 2735(EM1), 2736(EM1), 2737(PL1), 2738(PL1), 2739(PL1), 2740(PL1), 2741(EM1), 2742(ST2), 2743(EM1), 2744(WW1), 2745(WW1), 2746(SM1), 2747(WW1), 2748(SM1), 2749(WW1), 2750(WW1)[c], 2751(SM1)[c], 2752(SM1)[c], 2753(PL1)[c], 2754(ST2)[c], 2755(PL1).

c. Discussion of several papers, grouped by divisions.

AMERICAN SOCIETY OF CIVIL ENGINEERS

OFFICERS FOR 1961

PRESIDENT

GLENN W. HOLCOMB

VICE-PRESIDENTS

Term expires October 1961:
CHARLES B. MOLINEAUX
LAWRENCE A. ELSENER

Term expires October 1962:
DONALD H. MATTERN
WILLIAM J. HEDLEY

DIRECTORS

Term expires October 1961:
THOMAS J. FRATAR
EARL F. O'BRIEN
DANIEL B. VENTRES
CHARLES W. BRITZIUS
WAYNE G. O'HARRA
FRED H. RHODES, JR.
N. T. VEATCH

Term expires October 1962:
ELMER K. TIMBY
SAMUEL S. BAXTER
THOMAS M. NILES
TRENT R. DAMES
WOODROW W. BAKER
BERNHARD DORNBLATT

Term expires October 1963:
ROGER H. GILMAN
HENRY W. BUCK
EARLE T. ANDREWS
JOHN B. SCALZI
JOHN D. WATSON
HARMER E. DAVIS

PAST PRESIDENTS
Members of the Board

FRANCIS S. FRIEL
FRANK A. MARSTON

EXECUTIVE SECRETARY
WILLIAM H. WISELY

TREASURER
E. LAWRENCE CHANDLER

ASSISTANT SECRETARY
DON P. REYNOLDS

ASSISTANT TREASURER
LOUIS R. HOWSON

PROCEEDINGS OF THE SOCIETY

HAROLD T. LARSEN
Manager of Technical Publications

PAUL A. PARISI
Editor of Technical Publications

MARVIN L. SCHECHTER
Associate Editor of Technical Publications

IRVIN J. SCHWARTZ
Assistant Editor of Technical Publications

COMMITTEE ON PUBLICATIONS

THOMAS M. NILES, *Chairman*
WAYNE G. O'HARRA, *Vice-Chairman*

BERNHARD DORNBLATT
HENRY W. BUCK
JOHN D. WATSON
HARMER E. DAVIS